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ORDINARY MEETING.

3 November, 1936.

Mr. JOHN DUNCAN WATSON, the retiring President, in the Chair.

Mr. Watson said that he had the sad duty of having to announce that during the recess the members of The Institution had lost their Past-President and dear friend Sir Brodie Henderson. They had also lost Professor W. E. Dalby, Vice-President, Sir Archibald Denny, Bart., former Vice-President, Mr. K. A. Wolfe Barry, Member of Council, and H.R.H. Purachatra, Prince of Kambaeng Bejra, Siam, Honorary Member. Resolutions of condolence had been passed by the Council and transmitted to the respective families. In order to fill the vacancy among the Vice-Presidents caused by the death of Professor Dalby, the Council had appointed Mr. W. J. E. Binnie as fourth Vice-President for the remainder of the session 1935–1936, and Sir Clement Hindley as fourth Vice-President for the session 1936–1937.

On the recommendation of the Council, the members present elected by acclamation as

Honorary Members.

Field Marshal Sir William Riddell Birdwood, Bart., G.C.B., G.C.S.I., G.C.M.G., C.I.E., D.S.O., M.A., LL.D., D.C.L.

Sir William Henry Bragg, O.M., K.B.E., M.A., D.Sc., Sc.D., LL.D., D.C.L., F.R.S.

Charles Prosper Eugène Schneider, D.Sc.

Mr. Watson remarked that it was now his duty to vacate the Presidential Chair in favour of a very well-known member

of the profession. The members were to be congratulated on having chosen Sir Alexander Gibb as President of The Institution. Sir Alexander was one of the most active members of the Council, and had shown his determination to promote the interests of The Institution in the widest sense. Besides being an engineer, Sir Alexander was a business man, and realized that it was essential that a scientific institution, no less than a commercial organization, should be maintained on a sound footing. He was sure that the members would gain by having as their President the great-grandson of an Aberdeen engineer who had been an associate of Thomas Telford and had become a Member of The Institution in 1820.

Sir ALEXANDER GIBB, G.B.E., C.B., F.R.S., President,

having taken the Chair, thanked the members very much indeed for the honour they had done him by electing him as their President. He could only say that he would do his best to try to uphold the great traditions of The Institution.

Sir CHARLES MORGAN, Past-President, moved the following resolution :—

> " That the members present at this meeting desire, on behalf of themselves and others, to record their high appreciation of the services rendered to The Institution by Mr. John D. Watson during his term of office as President."

He was certain that it would be admitted by all the members of The Institution that they could not have had a more pleasant, amiable and desirable President than Mr. Watson. Engineers were of many types ; they had many different outlooks, and they were not easy to please, so that their President had to be tactful as well as efficient. Mr. Watson had been the most painstaking of Presidents ; he considered everybody's feelings, and had such a very happy way of doing so that those with whom he was dealing generally thought that they were getting their own way ! Mr. Watson had been a President worthy of The Institution and worthy of the members' best thanks.

Sir HENRY MAYBURY, Past-President, in seconding the motion, observed that Mr. Watson well deserved the kind remarks that Sir Charles Morgan had made.

The resolution was carried by acclamation.

Mr. WATSON, in acknowledging the vote of thanks, desired to say in the first instance how greatly honoured he felt by the members having passed the resolution. He accepted it with pride, even

though he did not feel worthy of it. To be President of The Institution carried with it great honour. That honour, however, could not be dissociated from heavy responsibilities and from the fulfilment of duties which even the most worthy and capable Past-Presidents had found to be neither easy nor light. All the members knew that the design and execution of works which were characterized as great engineering achievements were well-nigh impossible apart from well-organized team-work. That was also true when the work consisted of the organization and direction of the affairs of a great scientific institution. Had it not been for the unceasing and unswerving support and the brotherly help which he had received from his colleagues on the Council, he could not have done what the members had placed to his credit. The extension of the method of dealing with the business of The Institution by appointing committees to study and deal with specific subjects had imposed greater responsibilities on the committees, and particularly upon their Chairmen, but it had lessened the work of the President—a proceeding which became more and more desirable as time advanced and the duties of The Institution increased.

The Ordinary Meetings during the session had been well attended, the discussions had been good, but sometimes it had been difficult to allocate sufficient time for members to express their views on subjects under discussion. The speakers had been representative of even wider fields of engineering than usual, and had included non-members as well as members. He had had pleasure in attending the annual meetings of all the Local Associations, and he could not speak too highly of the excellent work which was being done by them. That autumn, for the first time, a Students' Conference had been held in Manchester, at which many representatives from various districts had taken part in discussing two excellent Papers, and had visited engineering works of outstanding importance. Unfortunately the esteemed Secretary of The Institution, Dr. Jeffcott, had not been able to attend the Ordinary Meetings, but his place had been well filled by Mr. Clark. Although handicapped by Dr. Jeffcott's enforced absence during part of the year, the staff had been all that could be desired and the work of The Institution had proceeded smoothly.

He thanked the Members once again for the kind resolution they had passed.

The PRESIDENT then delivered the following Presidential Address :—

GENTLEMEN,

For a century or so it has been the custom for your President on his assumption of office to address you on some aspect of Engineering. You have done me the honour of placing that duty on me for this evening, and I take as my subject " Engineers and Empire Development "—speaking in a way not so much to you, as on your behalf to, I hope, a far wider audience.

During many visits to the Dominions and Colonies I have had the opportunity of realizing and admiring the great engineering works that were the contribution of our predecessors to the Empire. To see such works in busy populous countries that even in the days of our grandfathers were vast empty spaces, is to understand the vital influence that the Engineer has had on the spread of civilization.

Ninety years ago, on the 20th January, 1846, Sir John Rennie, the third President of The Institution, made, on a similar occasion to the present, a very notable speech, calling it " a retrospective glance at the changes which have been effected in Great Britain since the days of that great man Smeaton." It was an epic speech ; in one hundred and three closely-printed pages he describes the rise of Civil Engineering in Great Britain from the year 1756, when Smeaton was first invited to design and build the new Eddystone lighthouse, to his own times when the railway age was well established. The period he reviewed, as is the period that has since elapsed, was just 90 years.

In 1724, when Smeaton was born, " There were," said Rennie, " no canals, railways, nor artificial harbours, or machinery, which at the present day would be thought worthy of the name ; and the public roads were little better than mere tracks across the country. . . . The inland transport was chiefly carried on the backs of *pack-horses*." England was still an agricultural country. In fact, with the exception of water-mills for grinding flour, there was no power-driven machinery anywhere. From the industrial and the manufacturing point of view Britain was wholly undeveloped, and as regards transport the country was not even as far advanced as 1000 years before.

By 1846 there had been a complete change. Britain had become the foremost nation in the world in wealth, power and prestige ;

the reason was given by Huskisson when, addressing in 1824 a meeting in London on the proposed memorial in Westminster Abbey to James Watt, he said: "Looking back to the demands which were made upon the resources of the country during the late war, perhaps it is not too much to say—at least it is my opinion—that these resources might have failed us before the war was brought to a safe and glorious conclusion, but for the creations of Mr. Watt and of others moving in the same career, by whose discoveries these resources were so greatly multiplied and increased. . . . But for the vast accession thus imperceptibly made to the general wealth of the Empire, we might have been driven to sue for peace, before, in the march and progress of events, Nelson had put forward the last energies of his naval genius, or, at any rate, before Wellington had put the final seal to the security of Europe at Waterloo." The great material expansion and development of Great Britain was due to "the creations of Mr. Watt and others moving in the same career."

The President's address of 1846 contains the story of these men and their works—of Smeaton, Watt, Telford, Rennie, Jessop, Chapman, Huddart, Murdoch, Maudslay, Miller and many others; "of the construction of roads, bridges, aqueducts, canals, river navigation and docks for internal intercourse and exchange"; "the construction of ports, harbours, moles, breakwaters and lighthouses, and in the art of navigation by artificial power for the purposes of commerce," the "construction and adaptation of machinery," the "drainage of cities and towns," and of all the other branches of Engineering born in that wonderful age, and so impressively referred to in the Charter of our Institution.

The world had apparently for centuries stood still; and then suddenly there arose a body of great engineers who, in a few years, created a new era. How did modern engineering rise so suddenly and so quickly blossom into great works and inventions? It was, I would suggest, a natural evolution from the more abstract discoveries of science, or natural philosophy as it would then have been called. From the middle of the sixteenth to the end of the seventeenth century there was a marked revolt of thought against prejudice and authority. In 1543 Copernicus published his great work. He was followed by many such as Galileo and Kepler; the new ideas reached the public mind through the works of men such as Bacon and Descartes. Newton's "Principia" was published by the Royal Society in 1687. The evolution went on; science and scientific thought were being established, and in due course there followed Engineering. So, at least, Engineers themselves thought—I believe rightly.

H. R. Palmer, actually our first member, said that "the

Philosopher" (that is to say, the Scientist) "searches into nature and discovers her laws and promulgates the principles upon which she acts. The Engineer receives those principles and adapts them to our circumstances. The working mechanic governed by the superintendence of the Engineer, brings his ideas into reality." Palmer, I need hardly remind you, was, so far as any one man can claim the credit, the founder of this Institution, and my quotation is from the Paper that he read at the first meeting of The Institution of Civil Engineers in the Kendal Coffee House on the 2nd January, 1818. From the outset Engineers thus looked on themselves as the practical exponents, for the ordinary use of mankind, of the scientific knowledge won by great scientists and mathematicians; and so they are still. "From research by itself," wrote Lord Rutherford in his 1934–1935 Report on the Department of Scientific and Industrial Research, "practical benefits can seldom be expected; processes, however promising in the laboratory, need development and method in their application if they are to be useful under factory conditions; and prosperity will only be restored when industry is prepared to carry the knowledge gained by research to a further stage and apply it to meet daily needs."

The basis and justification of the existence of the Engineer, and his contribution to civilization, exist in the fact that the application of science to practical use is in fact Engineering. Without the Engineer, I would claim, civilization as we know it could never have been achieved. This was the type of man that in Rennie's review was shown to have changed the whole outlook and life of this country. The same type of man, in the 90 years since 1846, has changed the face of the world and has created the British Empire. It is in no spirit of boasting that I would say that.

The marvels of Engineering have, in fact, surprised each generation in turn. Sir John Hawkshaw, addressing the members of this Institution in 1862, said "... the last twenty or thirty years have been the age of Engineers and Mechanicians. ... For in constructing railways, telegraphs, and steam-boats and their adjuncts docks and harbours, and moulding and fashioning the face of the material universe to the wants of man, in overcoming barriers, overleaping valleys and spanning seas, Engineers annihilate both space and time, bring into juxtaposition both nations and peoples, and accelerate, beyond all human expectation, personal communication, and that interchange of ideas which is all important to the advancement of civilization and knowledge.

"Distance and separation have led and will always lead to misapprehension and prejudice—to ignorance and mistrust ... and Engineers may feel, when labouring on the great public works that

facilitate the intercourse of nations, that they are not merely conquering physical difficulties, but that they are also aiding in a great moral and social work."

Again, only in last May wrote Mr. Vincent Massey, High Commissioner for Canada in this country, " Canada, in a very special way, may be considered the child of modern Engineering. The growth of land, sea and air transport during the last half century and, above all, the use of hydro-electric power in recent years have made it possible to build from the scattered colonies of British North America one of the foremost industrial nations of the world."

There could be no better instance of Engineering as a factor of nation- or Empire-building, and I would like to follow further along the lines of thought to which Mr. Massey's statement gives rise. The years since 1846 have been so packed with invention, development and exploitation that I would not attempt even to summarize in the briefest fashion the great names and the great deeds of the last two generations. I want, however, to remind you of some of the works of civilization that Engineers have carried out. All the instances that I shall quote are doubtless known to us here, but are not always sufficiently remembered and appreciated outside these walls. Let me say here that I speak of Engineers and Engineering in the widest sense—in the sense that the first civil engineers understood it—for which broad interpretation this Institution still stands.

Canals.—Historically one should, perhaps, in such a survey start with canals. " The opening up of the internal communications of a country," said Cobden, " is undoubtedly the first and most important element of its growth in commerce and civilization." " Hence," he adds, " our canals were regarded as the primary material agent of the wealth of Great Britain." This country in the eighteenth and early nineteenth centuries did indeed owe an enormous debt to the canals, which were then the sole means of transporting heavy goods over long distances, except in the few favoured areas served by navigable river-ways. They played a much less important part in Empire building. Hardly had they been introduced into the Colonies before they were for the most part superseded by railways. There are some notable exceptions. The first Welland canal was opened in 1829 ; the last but a few years ago. For 100 years there has been provided a navigable waterway to surmount the 325-foot rise between lake Ontario and lake Erie, and now with the system of canals on the St. Lawrence and at Sault St. Marie, it is possible for ocean-going steamers to penetrate half-way across the Continent of America, over 2,250 miles from the sea. Even that is not the end. With restless ambition Canadian engineers have planned still greater expansions of their inland waterway system which, if carried out, may well

have a vital and perhaps unpredictable effect on the whole future of Eastern Canada.

Two other ship-canals, although only partially connected with British engineering, have had a profound effect on the British Empire: these are the Suez and the Panama Canals. The Suez Canal shortened the journey to India and the East by 3,750 miles, and is perhaps the main artery of the British Empire. The Panama Canal shortened the ocean journey to Vancouver by 6,000 miles.

Much of the time spent by our predecessors on canal-building seems now to have been a pathetic and unhappy waste of skill, energy and money; but we must not let our judgement be obscured by a knowledge and experience which is ours, but which was not theirs. And, illusory though such a project as the Mid-Scotland Ship-Canal may now seem, it would have been worth a hundred million pounds to have had it during the Great War.

Roads.—Roads and Empire are coeval. After the break-up of the Roman Empire its great road-system fell into decay, not only in this country but generally throughout Europe. Even when centuries later development and progress came again, road-making was not recognized as a necessity. Smeaton was despised by his fellows for undertaking road work, then still looked upon as beneath the dignity of the Engineer and as affording no scope for his skill, until Telford and Macadam revolutionized ideas.

How very near they came to anticipating the present road age is not always realized. Chance and circumstance, vested interests and private greed combined to postpone for 70 years or so the development of a coherent road policy, such as Telford almost initiated. In the 1820's it was still quite possible that steam transport would be primarily a matter of road transport. Very considerable advance in this direction was made, but ultimately the railways—quite properly—won. When roads, after a long period of inertia, came into their own again, the internal-combustion engine, as applied to road carriages, started in the 1890's just about where Dance's and Gurney's and other steam carriages had left off more than 100 years ago.

Roads have now, I suppose, succeeded to the first place as the main arteries of any country. Equally they must be almost the first engineering project in any new settlement or colonization. To many of us the days of Livingstone and "Darkest Africa" seem but a short time ago, and yet to-day a great road linking South and East Africa runs through country that he explored and provides a popular route for touring motorists.

More than 1,000,000 miles of modern roads now serve the Empire. Even yet we are only on the threshold in many countries. In

Kenya, out of 10,500 miles, only 723 have a metalled surface, the remainder being earth-roads, impassable after heavy rain. In the 370,000 square miles of Nigeria, more than half of the 15,000 miles of road are fair-weather roads only. I have selected these two examples at random, merely to give some idea of the immense amount of work that still awaits the road engineer. Looking farther ahead, consider what will be the ultimate social and political effects in such countries as these, when road-planning has been carried out on really imperial lines, on the model, for instance, of the Pan-American Highway from Alaska to Cape Horn. The Roman roads were essentially the symbols of Empire and prosperity. To anyone who has seen the Italian motor roads, their successors to-day, it is clear that the Age of Roads has returned.

Bridges.—" The bridge," it has been said, " even more than the road, is a symbol of man's conquest of nature." Certainly some of the most wonderful engineering works of all ages have been bridges, not only from an æsthetic or a technical point of view, but from a cultural and civilizing aspect, too. Essentially, a bridge carries traffic over a gap that would otherwise be impassable. Its effect, therefore, cannot fail to be great. Without bridges roads must deviate for many miles to find a means of crossing in the higher reaches of the river. Even for the Romans, river fords were the fixed points on their lines of communications. Fortunately fords no longer interest us. Ferries we still have, and, with all the delay, the inconvenience, cost, and danger that they involve, they are in some cases still a valuable link in communication. Obviously, however, the bridge is the only ultimate solution.

As recent examples of the effect of individual bridges in opening up and changing the economy of a whole country, one cannot do better than instance the 12,064-foot long Lower Zambezi bridge, the longest bridge in the world, opened in 1935, or the Birchenough bridge over the Sabi river in Southern Rhodesia, which reduced from 600 miles to 450 miles the route between Buluwayo and Beira, and makes readily accessible for the first time a wide area in the east of Southern Rhodesia.

We are entering into an age of immensely large bridges. There is technically almost no limit to what the Engineer can bridge, which means incidentally that one of the most effective of natural boundaries—the river—now ceases to be of material import.

Railways.—Railway construction in the Empire may be said to date from the middle of last century ; the first railway was opened in about 1850 in Canada and Australia, in 1853 in India, and in 1860 in South Africa. In 1850 Canada had 66 miles of railway ; she has now 43,000 miles, whilst India has about the same. Australia has

nearly 30,000 miles. In such vast countries, the railways have been for many years, and still are, the only through transport routes. They afforded the only means by which great continental areas could be opened up. Without railways, trade and industry must have been confined to the coastal areas or to the banks of navigable rivers. The possibility of penetrating into the interiors at once changed our outlook in regard to these great land areas. The old colonial system had been based on islands or coastal towns. Roads, canals, but chiefly railways, altered all this, permitting the far-reaching emigrations of the nineteenth century that peopled the new world, and making possible the development of the mineral and other natural resources of the farthest hinterlands.

I might quote two striking instances, well-known though they are. The Indian railway systems, long before they were completed, banished for the first time in that world-old country the ever-present spectre of famine. The Canadian Pacific Railway—perhaps the greatest of all individual works of empire-making—brought a new ocean into the Empire. Vancouver has recently been celebrating its jubilee. In 1862 there was a log cabin and three men on Burrard Inlet. To-day Vancouver has a population of over 300,000, has shipped over 100,000,000 bushels of wheat in a single year, and through the Canadian railway systems, is the Pacific outlet for an area of 1,000,000 square miles.

It was not only in the British Empire that British engineers built railways; in fact, railway building was perhaps the greatest single factor in the unexampled prosperity of this country in the last century. British railway engineers extended their activities to every country in the world, and it is not necessary for me to emphasize what in the end that has meant, and how profoundly it has changed the outlook, life, politics, and industry of every people in the world.

Ships.—Next to railways in this respect comes, I suppose, shipbuilding and ship-owning. The steamship era came into being more or less simultaneously with the railways. The steam engine produced the Industrial Age with all its requirements, aggregations of population, the carrying of increasing volumes of food, raw materials and manufactured products. Soon it became necessary to look overseas for raw materials and markets. The population grew enormously, until in time we came to depend on foreign sources for a large part of our food supplies, and to look abroad for a home for our surplus numbers. In all this the steamship formed the link, a link as necessary as the bridge is to the road.

Development has been amazingly rapid. In this the year of the maiden voyage of the "Queen Mary" it is striking to recall that

60 years ago the Cunard Company still possessed a paddle steamer for the Atlantic service, the old "Scotia."

It was the application of steam and steel to ship-building that has enabled this country for 80 years to be the greatest ship-builder and ship-owner in the world. Before the War the Empire owned nearly half the world's tonnage, it still owns a third. We dominated for years the Seven Seas, and it was because of that that our young Dependencies and Colonies could pass, peacefully and untroubled, the early difficult years of development and growth.

As another example, the invention of cold storage made possible a trans-oceanic trade in meat, fruit and all sorts of most perishable foodstuffs, on which we now depend for life and our Dominions for a livelihood. Within 3 years of its introduction, Victoria and New South Wales had built up with Great Britain a yearly butter trade of £1,000,000. In 1935 Great Britain imported over 15,500,000 hundredweights of fresh fruits from the Empire, and nearly every sort of fruit is now obtainable in London in every month of the year. The sea-borne meat trade means to Empire farmers some £80,000,000 a year.

Ports.—Increasing changes in ships and in the business of shipping during the last century have had their effect on port engineering, and it has sometimes seemed that the demands of shipping threaten to impose burdens on ports that, while not beyond the Engineer's technical skill, may cease to be economic. With maximum draughts up to 40 feet, the largest Atlantic liners can now berth in less than a dozen of the major ports of the world, and only very heavy expenditure has made even this possible. It was, I might add, near a point on the Clyde where Smeaton's old chart shows a depth of 3 feet 6 inches at high tide that the "Queen Mary" was launched in 1934.

As a great part of all cargoes is now being carried in freight liners that run to definite schedules, with heavy penalties for delays, continuous expenditure is being required on port facilities, cargo handling-arrangements and intensive mechanization of every kind. The equipment and facilities of a modern port are—like its organization and management—on so large a scale nowadays, that a port, such as London, is a miniature empire in itself. These developments have gone on increasingly, and it is still impossible to see when a halt may be called, although, as I have hinted, the Engineer must look to the economic as well as to the technical aspect.

Transport.—I have referred to the major means of transport on land and sea, on the success of which modern civilization, with its increasingly artificial conditions, is dependent. At no time has Engineering, I think, failed technically to keep abreast of growing demands, although to a great extent, the demands themselves have

grown out of the opportunities that the engineer creates. There is, indeed, a danger that the machine may take charge, and perhaps the greatest problem that the Engineer now has to face is the sane control of the forces of Engineering. Nowhere is this more obvious than in transport. Co-ordination of transport has been started in this country, but has not yet got very far. It seems to me certain that sooner or later we must come to something like a unified control, at least in all questions of broad policy, of all means of transport.

Obviously, before I leave Transport, I must refer to aerial transport. Man has always longed to fly, but not until the invention of the internal-combustion engine was it practicable. It is just over 30 years since the Wright brothers made their first successful flight, and only some 25 years since Bleriot first flew across the Channel. Progress has never ceased to be rapid, but during the last 10 years it has been so amazing that we have come to take for granted one epoch-making achievement after another. Spectacular flights at 6 miles a minute; the record of Campbell Black and Scott in reaching Australia on the third day out from England; Flight-Lieutenant Swain's recent flight at a height of nearly 10 miles, are not isolated or wasted instances of heroism and skill, but practical steps in the economic and commercial exploitation of the air. It will clearly become quite usual in a year or two to breakfast in London and to dine the next day in Delhi or Montreal. Imperial Airways now fly an average of 17,000 miles a day, in four continents and twenty-four different countries. They carry annually over sixty thousand passengers and fifteen million letters.

The commercial application of the aeroplane has not yet been developed very far, but it was almost wholly by air transport that the New Guinea goldfields, and various gold and copper mines in northern Canada, have been opened up. In New Guinea, all the machinery and parts for two large dredgers and a hydro-electric plant of several thousand horse-power were carried wholly by aeroplane, over a range of mountains 5,000 feet high, into the interior, and then assembled and put to work within a year.

Before finally leaving the subject of Transport, I would just refer to the development of oil pipe-lines as a means of oil-carriage. The pipe-line from the Iraq oil-fields to the Mediterranean, itself a wonderful engineering feat, almost completely upsets the ideas of geography that one learnt at school!

Agriculture.—Without, I hope, wearying you, may I very briefly mention a few other directions in which Engineering has advanced civilization and created the Empire. The droughts of the Middle West of the last few years have, perhaps, caused some doubts as to the ultimate result of turning the western provinces of North America

into the greatest grain-growing area in the world. New ideas and inventions are all sooner or later pressed a little too far, and that may have happened there to some extent. Broadly and basically, however, it was creative civilization in its best form that brought into cultivation in western Canada 25,000,000 acres of virgin soil which now produce over 400,000,000 bushels in a year; this result was rendered possible only by the adaptation of engineering inventions at every stage, in the forms of the plough, the harvester, the railway, the grain-elevator, and the ship.

Irrigation.—The effects and successes of irrigation are perhaps even more striking. It is in principle as old as man, but, with the exception of one or two localities, it is within the last two generations only that it can be said to have had anywhere more than purely local significance. After the notorious Indian famine of 1865–67, irrigation works were started there on modern lines. That famine affected an area of 180,000 square miles with a population of forty-seven million people. The great famine of Southern India in 1876–78 involved 257,000 square miles and fifty-eight million people, of whom five and a half millions died. Famines do not now occur in India; shortage and scarcity exist at times, but the possibility of areas of the size and population of Great Britain being left without food owing to failure of a capricious monsoon has long since been ended. In 1931 over 31,000,000 acres of cultivated land were irrigated by Government irrigation works in India, at a capital cost of about £102,000,000. The yearly value of the crops reaped from the irrigated land is about £65,000,000.

The latest, and perhaps the most spectacular, of Indian irrigation projects is the Lloyd Barrage across the Indus at Sukkur, built at a cost of over £15,000,000. It has more than doubled the area of cultivated land in the whole Province of Sind. It, too, like many great engineering works, is having its difficulties such as are inseparable from major interferences with nature, but no one doubts that the Engineer will win in the end.

In speaking of irrigation, mention of Egypt should not be omitted. Since the British connection began in 1882, the repair and extension of the old irrigation system has more than doubled the productivity of Egypt, and has resulted in the growth of the population from under seven millions in 1882 to over fifteen millions to-day.

I would, in passing, refer to another type of irrigation in the Great Australian Basin of 600,000 square miles, where, as there are no rivers, over three thousand artesian wells now tap the subterranean water, giving a daily flow of 500,000,000 gallons.

Water-Supply, Sanitation, etc.—I now turn to domestic water-supply, so much to the fore in this country in the dry years of 1933–35.

It is well to remember that even the worst cases of water-shortage then so constantly quoted would have been the usual experience in most parts of this country in our fathers'—certainly in our grandfathers'—time. It was not until after 1866 that cholera, except as an occasional epidemic, was eradicated from this country. Over fifty-three thousand people died of it in 1848; over twenty thousand in 1854.

Many whose experience of Indian conditions is by no means remote probably find it hard to realize that even there clean water and sanitation are really becoming matters of practical import—truly one of the major social revolutions in the world! Large centres of population, wherever they may be, are no longer necessarily centres of disease. One could refer here, too, to such important aspects of Engineering as drainage and reclamation, but time passes.

Other Branches of Engineering.—The Institution of Civil Engineers stands for every form of engineering, and for all who turn the resources of nature to the use and benefit of mankind. So vast has been the extension and so many the ramifications of engineering in these latter days, that it would be quite impossible to follow up all these diverse paths, and the mechanical engineer, who has indeed been closely concerned in most of the instances I have quoted, must forgive me if I do not more specifically refer to the inventions and progress for which he has been responsible. The same applies to younger branches, such, as for instance, chemical engineering, a development of the last 20 years and one of the bases of modern industrial planning.

I must, however, devote a little time to some aspects of the greatest engineering development of the latter part of the nineteenth century, namely, the applications of electricity, including the telegraph, the telephone, wireless in all its forms, electric light, heating and refrigerating, electric railways, electric furnaces, electric motors, electro-chemistry—itself a vast subject—and so on.

Within two generations the telegraph and telephone have become the principal channels of business communication. The development of wireless is within the memory of all. Many of us had already started on our engineering careers when the first electric railway, the City and South London tube, was opened. By radio the most distant parts of the world are divided in point of time by the smallest fraction of a second only. Space, at least within the confines of this small planet, is ceasing to have any meaning.

In another field of electrical engineering, the profound effects of the large-scale development of power from water has already been referred to. Industry is no longer chained to coalfields. Adverse though it has been in some ways to our own narrower national

interests, the rise of hydro-electricity has been one of the most powerful influences in Empire-development. The most spectacular example of all is Canada. The total installed power there is 8 million horse-power, more than four-fifths of which is in the industrial provinces of Ontario and Quebec, where there is no coal. More than 95 per cent. of Canada's generating plant is water-operated.

So much for examples; they are but sketchy. I hope I have, however, sufficiently illustrated the two points with which I started —that Engineering, turning to practical account the discoveries of Science, is the basis of all civilization, with all its achievements as well as its faults; and that in the same way Engineering has been the foundation of our own great Empire—as it must be of every Empire.

These are imposing claims to put forward, and serious responsibilities to assume, but they are, I feel, justified. If this high position has not always been accorded to us Engineers by popular opinion, it is, I fear, in some degree because we ourselves have not always fully realized it. Although I am only speaking as the President of The Institution of Civil Engineers, I would humbly claim that the whole history, intention, and objects of our Institution justify me in speaking on such a matter for all Engineering in this country.

I have failed in my intention if I have not demonstrated what Engineering has done in moulding the history of the World and of our Empire. In my opinion the opportunities of the future are vastly greater than any that the past has offered, but, frankly, I look with anxiety on the years to come. The machine, as I have already said, sometimes seems to be taking control. Inventions and developments succeed one another with bewildering speed, and there seems, unfortunately, to be no limit to the possible results of uncontrolled and misapplied ingenuity. In such circumstances no one can say where Engineering may lead us or what limit there is to the power of the Engineer. One thing is certain, and that is that there must be control.

This is an age of ever-increasing specialization. The ramifications of engineering enterprise are endless. After all, Engineering, I estimate, provides directly or indirectly the livelihood of about one-seventh of our working population. It is inevitable that Engineering should have become split up into dozens of different categories and groups. There are in this country alone over a hundred reputable engineering institutions and societies, with their own objectives and supporters. Sometimes we seem to lack that sane outlook and wider vision that characterized the outlook of our own Institution in its earlier days. If, however, we are to be able to deal adequately with

the great problems of the future, we should, I am certain, seek now to put a brake on this continuous disintegration, and should attempt in some form or other to co-ordinate and unite engineering activities in the broadest sense.

This is a view on which I have, I believe, the support of many eminent engineers. It is not my intention here to attempt to detail means of attaining the objective that I put before you, but I would emphasize as strongly as I can the necessity for all of us, individuals and Institutions alike, to subordinate some of our more personal and independent views and feelings to a common policy.

This Institution, I am sure—and in this again I think I can speak for my colleagues—will always be ready to do all that is possible to achieve such an objective. In recent years, as many of you know, not a little has already been done to prepare The Institution for a new period of even greater activity than it has yet known. Its powers, scope, and objects were closely investigated and considered by a specially-appointed Committee. In the successful expansion of its activities in connection with research, The Institution has lately undertaken a number of enquiries of great importance to Engineers. These are evidences of the re-orientation in our outlook that is taking place.

Useful work has also been done in the wider question of co-operation with other Engineering Institutions, but as yet it is still on far too restricted a scale. I would like to implant in the mind of every engineer in this country the idea of co-operation. I would like it to be possible for one broad policy to inspire and guide all classes of engineer. I would hope that in time there would arise a body of engineering opinion so weighty, so authoritative, so sure, so sane, that it would prevent waste of energy and misplaced enterprise, and would inevitably command attention in the politics and administration and life of our country and our Empire. I believe that that would be the greatest—and perhaps the only—safeguard for the future of civilization.

Sir HARLEY DALRYMPLE-HAY moved :—

"That the best thanks of The Institution be accorded to the President for his Address, and that he be asked to permit it to be printed in the Journal of The Institution."

In doing so, he remarked that he had known Sir Alexander Gibb for a great many years, and had been very delighted when a gentleman of Sir Alexander's experience had been proposed for the Presidential Chair. Some of the members might not know that Sir Alexander had started his professional career with a distinguished firm of

consulting engineers, Sir John Wolfe Barry & Partners. He had then taken up a position with one of the largest contracting firms, of which his father had been a principal. Having gained very wide experience in contracting, he had then returned to consulting engineering, having thus had a very valuable dual experience. In his opinion, therefore, the members had indeed been wise when they had chosen Sir Alexander as their President. Sir Harley hoped that in the future there might be found in the Presidential Chair some distinguished engineering contractor. After all, an engineering contractor was an engineer, and occupied a very important position, especially in view of the remark often heard amongst engineers, "We will leave that to the contractor"!

Mr. F. M. G. Du-Plat-Taylor seconded the motion, which was carried by acclamation.

The President thanked the members for the kind way in which they had carried the vote of thanks, and for the patience with which they had listened to his Address. He would have great pleasure in permitting his Address to be printed in the Journal of The Institution.

He wished to take the opportunity of thanking the guests who had honoured the meeting with their presence that evening. There were present the Presidents of the Institution of Electrical Engineers, the Institution of Gas Engineers, and the Institution of Water Engineers, and he desired to thank them especially.

MEDALS AND PREMIUMS.

The PRESIDENT presented the Coopers Hill War Memorial Prize and the James Forrest Medal, and the Awards for Session 1935–36 were announced, as follows :—

FOR PAPERS READ AND DISCUSSED AT THE ORDINARY MEETINGS.

The following, being members of Council, are ineligible to receive awards for their Papers, and the Council have expressed to them the thanks of The Institution :—

David Anderson,[1] LL.D., B.Sc., for his Paper on "The Construction of the Mersey Tunnel";

Sydney Bryan Donkin, for his Paper on "Industrial, Agricultural and Domestic Heating with Electricity as a By-Product";

Sir Robert Abbott Hadfield,[2] Bart., D.Sc., D.Met., F.R.S., for his Paper on "The Corrosion of Iron and Steel," of which he was co-Author with Sidney Arthur Main, B.Sc.; and

Reginald Edward Stradling, C.B., M.C., Ph.D., D.Sc., for his Paper on "Road Engineering Problems: Judging the Slippery Road," of which he was a co-Author with Reginald George Cyril Batson, M. Inst. C.E., and George Bird, B.Sc.

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1. A Telford Premium and the Coopers Hill War Memorial Prize for 1935–36 to Ernest James Buckton,[3] B.Sc., and Harry John Fereday, MM. Inst. C.E., jointly, for their Paper on "The Demolition of Waterloo Bridge."
2. A Manby Premium to Sidney Arthur Main, B.Sc., for his Paper on "The Corrosion of Iron and Steel," of which he was co-Author with Sir Robert Hadfield, Bart.
3. A Telford Premium to William Henry Glanville, D.Sc., Ph.D., M. Inst. C.E., Geoffrey Grime, M.Sc., and William Whitridge Davies,[4] B.Sc., Assoc. M. Inst. C.E., jointly, for their Paper on "The Behaviour of Reinforced-Concrete Piles during Driving."

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[1] Has previously received a Telford Gold Medal and a Telford Premium.

[2] Has previously received a Telford Gold Medal, a George Stephenson Gold Medal, a Howard Quinquennial Prize, and a Telford Premium.

[3] Has previously received a Telford Premium.

[4] Has previously received a Miller Prize.

4. A Telford Premium and the Indian Premium to Wilfrid Cracroft Ash,[1] B.Sc. (Eng.), and Oscar Branch Rattenbury,[2] B.Sc. (Eng.), MM. Inst. C.E., jointly, for their Paper on "Vizagapatam Harbour."

5. A Telford Premium to Duncan Kennedy, M. Inst. C.E., and Hubert Edward Aldington, Assoc. M. Inst. C.E., jointly, for their Paper on "Royal Docks Approaches Improvement, London."

6. A Telford Premium to Shewell Reginald Banks, M.Eng., Assoc. M. Inst. C.E., for his Paper on "The Superstructure of the Island of Orleans Suspension Bridge, Quebec, Canada."

7. A Telford Premium to Professor John Fleetwood Baker,[3] M.A., D.Sc., Assoc. M. Inst. C.E., for his Paper on "The Rational Design of Steel Building Frames."

8. A Trevithick Premium to Arthur Henry Barker, B.Sc., B.A., M. Inst. C.E., for his Paper on "A General Comparison of Gas and Electricity for Heat-Production."

9. A Trevithick Premium to Frank Sturdy Sinnatt, C.B., M.B.E., D.Sc., for his Paper on "Some Major Problems in the Utilization of Coal."

10. A Telford Premium to Reginald George Cyril Batson,[4] M. Inst. C.E., and George Bird, B.Sc., jointly, for their Paper on "Road Engineering Problems: Judging the Slippery Road," of which they were co-Authors with Dr. Stradling.

For Papers Published Without Oral Discussion.

1. A Telford Premium to Jens Peter Rudolf Nielsen Stroyer, M. Inst. C.E., for his Paper on "Earth Pressure on Flexible Walls."

2. A Telford Premium to Powys Davies,[5] M. Inst. C.E., and Shivram Vasudeo Puranik, B.E., jointly, for their Paper on "The Flow of Water through Rectangular Pipe-Bends."

3. A Crampton Prize to the late Edward Thomas Mervyn Garlick, M. Inst. C.E., for his Paper on "Sedimentation and Anaerobic Digestion of Sewage-Sludge at Colac, Echuca, and Mildura, Australia."

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[1] Has previously received a Telford Premium and a Trevithick Premium.
[2] Has previously received a Bayliss Prize and a Miller Prize.
[3] Has previously received a Telford Gold Medal.
[4] Has previously received a Telford Premium and a George Stephenson Gold Medal.
[5] Has previously received a Telford Premium.

For Papers Read at Students' Meetings in London and by Students before Meetings of Local Associations.

1. The James Forrest Medal and a Miller Prize to Edwin Lomax, Stud. Inst. C.E., for his Paper on "Modern Swimming-Pool Design."
2. The James Prescott Joule Medal for the triennial period 1933–1936 to Ronald Bridgman,[1] Stud. Inst. C.E., for his Paper on "Modern Permanent-Way Design."
3. A Miller Prize to Denis Frank Orchard, Stud. Inst. C.E., for his Paper on "The Storström Bridge."
4. A Miller Prize to Leslie Charles Waters, Stud. Inst. C.E., for his Paper on "Notes on the Design and Construction of Concrete Roads."
5. A Miller Prize to Derek George Thomas, Stud. Inst. C.E., for his Paper on "Installation of Escalators and Reconstruction at Moorgate Station."

Bayliss Prizes.

Bayliss Prizes, awarded on the results of the October, 1935, and April, 1936, Examinations, respectively, to John Woollhead Hooper, B.Sc., Stud. Inst. C.E., and Kenneth Frederick Geesin, Stud. Inst. C.E.

Charles Hawksley Prize.

No award of the Charles Hawksley Prize for 1936 has been made, but having regard to the good work shown by two of the competitors, namely, Geoffrey Wood, B.Sc., Stud. Inst. C.E., and James Alexander Chew, Assoc. M. Inst. C.E., they have received honourable mention and have been granted £50 each.

[1] Has previously received a James Forrest Medal and a Miller Prize.

Paper No. 5045.

"The Restoration of the Breach in the Right Guide Bank of the Hardinge Bridge."

By Bertram Lionel Harvey, O.B.E., B.Sc., M. Inst. C.E., M.I.E. (Ind.).

(Ordered by the Council to be published with written discussion.)[1]

TABLE OF CONTENTS.

Introduction.

The Hardinge bridge over the Lower Ganges river at Sara in Bengal consists of fifteen main spans of 350 feet, with three land spans of 75 feet at each end. The piers of the bridge are built on wells, which have semicircular ends and straight sides, with two dredging holes; the wells were sunk to 150 feet below low-water level, the steining being of concrete blocks with a mass-concrete core. The depth to which these foundations were sunk was fixed in relation to the deepest scour, which, after many years of surveying and investigation in the vicinity of the bridge site, was found to be 100 feet below low-water level. The additional 50 feet beyond this depth was considered as adequate for safety, especially as the bridge was

[1] Correspondence on this Paper can be accepted until the 15th March, 1937, and will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

sited for a straight flow past its piers, and scour greater than 100 feet below low-water level could not be expected. The maximum recorded discharge of the river at the site of the bridge was 2,000,000 cusecs and the maximum calculated discharge was 2,500,000 cusecs. On this figure the bridge was designed with a clear waterway, deducting the obstruction from guide-bank slopes and piers, of 4,687 feet.

As the surrounding country is of very low formation, it was considered preferable to add 200 feet to the datum values and thus avoid any of the working levels being in minus figures; consequently highest flood level (in relation to the mean sea-level at Karachi) is R.L. 250·00 instead of R.L. 50·00, and similarly lowest water level is R.L. 219·00 instead of R.L. 19·00. The general level of the bottoms of piers on this basis is R.L. 69.00, except for piers Nos. 2 and 15 which, by reason of their proximity to the guide banks, were taken 10 feet deeper, to R.L. 59·00.

The main protection works of the bridge, that is to say, the guide banks for retaining the river within the limits of the bridge, were designed on the Bell bund principle with certain modifications arising from local conditions. The river in its wanderings during the last two centuries across these alluvial plains has been held at two places where there are deposits of a black practically-inerodible clay, and as these were really controlling points in the regime of the river, about 4,000 feet of the river bank at each place was protected by a revetment of stone boulders to ensure its permanence. These two places are at Raita on the right bank, 11 miles upstream by the present channel, and Sara on the left bank, 3 miles upstream of the bridge. These two natural controlling points having been made permanent, the guide banks flanking the bridge were made 4,000 feet in length, of which 3,000 feet, or approximately three-fifths of the width of the waterway of the bridge at low-water level, was upstream of the bridge. The existence of the two outlying fixed points at Raita and Sara was thus taken into consideration in the design of the guide banks at the bridge and in the protection scheme as a whole.

The body of the guide banks was of sand, the slope on the river side being covered, first by a 9-inch layer of good stiff clay, and then by a 3-inch layer of quarry chips on which the facing of boulders was laid. At the foot of the slope just above low-water level an apron of loose boulders was provided on the floor of the borrow pits from which the sand for the guide banks was dug, so that, when scour occurred at the toe of this apron, the boulders would fall in and eventually, when the full scour of 100 feet below low water had been reached, there would be a continuous slope of

boulders from the top of the guide bank, in continuation of the stone on the face, down to the bed of the river.

The quantity of stone to give this result was found by a formula based on the recommendations of the late Sir Francis Spring, K.C.I.E., M.A.I., M. Inst. C.E., at one time Chief Engineer to the Public Works Department of India, in his Paper published in 1903.[1] The formula is :—

$$Q = 2{\cdot}25\ T\ (R + F) + 2{\cdot}82\ DT$$

where

Q denotes the quantity of stone required in cubic feet per foot run,
T „ thickness of stone laid on the slope,
R „ difference between high-flood level and low-water level,
F „ freeboard between high-flood level and the top of the bank, and
D „ deepest known scour below low-water level.

This formula has been adopted in the design of guide banks of many bridges.

In the case of the guide banks of the Hardinge bridge, with a deepest known scour of 100 feet below low-water level, the quantity of stone laid was approximately 1,260 cubic feet per foot run, which was distributed as follows : 238 cubic feet on the slope of the constructed bank, and the balance in the apron in strips, 50 feet wide and 5 feet 6 inches, 7 feet 6 inches and 8 feet 6 inches in depth, making up the total of 1,258 cubic feet per foot run.

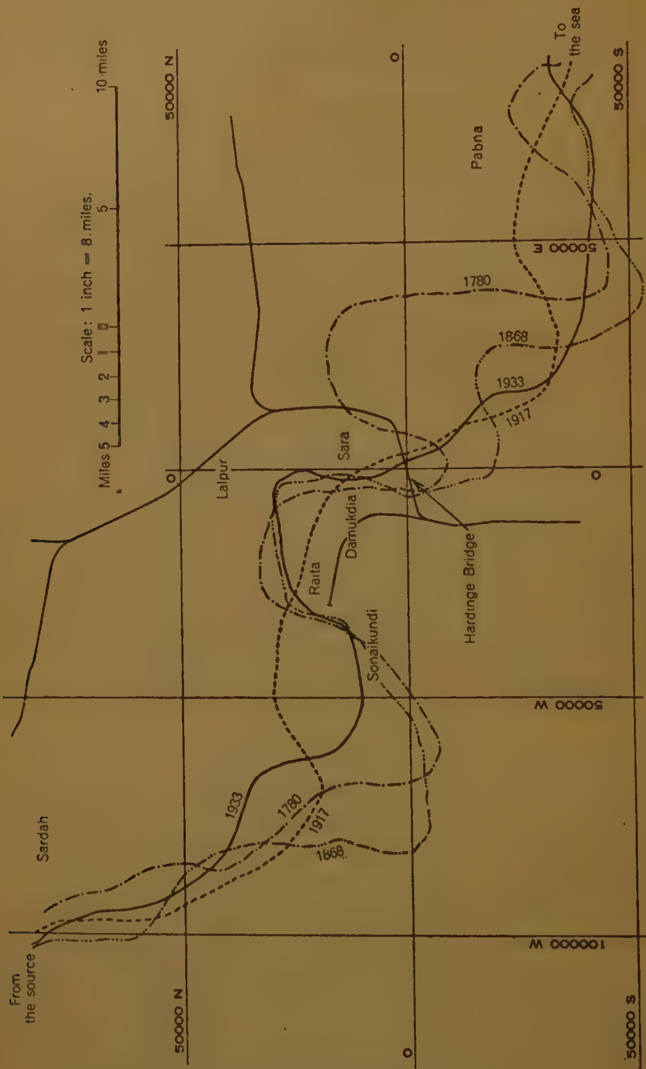
The River Ganges.

The river Ganges, from its source on the southern slopes of the Himalayas, flows in a south-easterly direction between more or less defined banks, within which it has flowed for centuries, for a distance of about 1,100 miles before it arrives at the alluvial plains of Bengal, below the Rajmahal hills. From there it has during the more recent centuries flowed in innumerable courses and channels. *Fig. 1* indicates the channels it has occupied at four different periods, and this alone should show the difficulty engineering science has had in so "training" the river as to ensure that it will always, in spite of its wandering elsewhere in its course, flow through a bridge that has been built across it. The movement of these channels

[1] "Indian River Training and Control," p. 48 and Plate XX. Technical Paper No. 153, Simla, 1903.

from one alignment to another is in fact sometimes a very rapid process, and even in the short period of 3 years that the Author has been directly connected with the problems at the bridge, very

Fig. 1.



GENERAL MAP.

noticeable changes in curvature and alignment of the stream have occurred.

Fig. 1 also shows that the two controlling points referred to,

namely at Raita and Sara, are at the downstream ends of the two large bights which are the limits that the river has occupied in past years. These are the Sonaikundi bight upstream of Raita, and the Lalpur bight upstream of Sara. When the bridge was completed in 1915, the main channel of the river and that part of it which flowed only in the low-water season was hard up against the left bank and the left abutment of the bridge. When the river occupied this position, the north end of the Sara protection bank, flanking the Lalpur bight, was well inland from the water's edge; but in the year 1925 the main river, moving eastwards into the Lalpur bight, came into contact with the unprotected river bank northward of the Sara revetment. This bank did not provide the resistance expected of the Sara clay and, in the years that followed, the embayment increased and, in the year 1931, the upper end of the revetment, to which large quantities of stone had been added, was enveloped, isolated and sunk during the flood season. The sweep of the river into the embayment thus formed, assisted by the eddies caused by the obstruction of the sunken work, directed towards the right bank a current which threatened a serious embayment about 2 miles above the bridge. The eddy downstream of the sunken work was found to have a diameter of 800 feet and soundings showed a depth of 170 feet below low-water level. In these circumstances the matter was referred to the Consulting Engineers, Messrs. Rendel, Palmer and Tritton, and on their advice in 1933 the protection bank was curved back to promote smooth flow, to avoid further damage to the bank and to minimize or prevent altogether the dangerous eddies, whilst the Damukdia guide bank was constructed.

The Damukdia Guide Bank.

Although Punjab rivers have been allowed, under Bell's[1] method of river-training, to loop in behind the guide banks in the expectation that a cut-off would bring the river through the bridge, no such experiment could be contemplated with a river of the character and magnitude of the Ganges in its course through the delta, and the construction of a guide bank near Damukdia to check the threatened embayment was necessitated. The Damukdia guide bank was aligned on to the middle of the fourth span of the bridge and was made 4,000 feet long; its apron was designed to meet a

[1] J. R. Bell, "The Continuous Bund and Apron Method of Protecting the Flanks of Bridges for Rivers in the Punjab (India)." Technical Paper No. 2b, Simla, 1890.

scour of 160 feet below low-water level for the downstream half of its length, and a scour of 100 feet for the upper half of its length. The quantities involved in the work were nearly 2 *crores* (or 20,000,000) cubic feet of earthwork and 90 *lakhs* (or 9,000,000) cubic feet of boulders (weighing on an average 85 lbs. each and commonly known as " one-man-rock ").

The cost of this and other protective works done in the year 1933 was approximately £150,000. It should be mentioned here that, although the bend scour for which the guide banks of the bridge had been designed was only to 100 feet below low-water level, pot-holes had been sounded at Sara to 170 feet depth and, although these were the result of the spur action and not of bend scour, it was decided that for the Damukdia guide bank, and for future works in the vicinity, all aprons should be designed to meet a scour of 160 feet depth. The Damukdia guide bank was completed in 7 months, on the 1st June 1933, and a feeling prevailed that all was well with the bridge and that the danger of its isolation by the river advancing further westwards in this vicinity had passed for ever.

Causes of the Breach of the Right Guide Bank.

The behaviour of the river was normal during the rise of the year 1933, but, unfortunately, it was not normal during the fall.

Fig. 2.

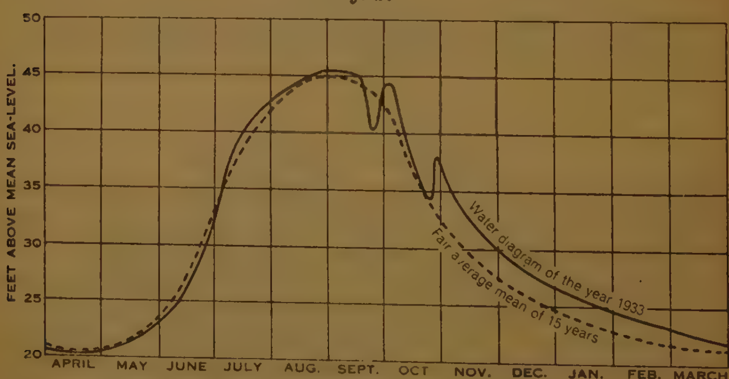


Fig. 2 shows the average hydrograph of the river as well as the diagram for the year 1933; this brings out the fact that the main fall normally commences early in October. In 1933, however, the main fall appeared to commence early in September; this was attributed to the failure of the monsoon, and it was concluded that,

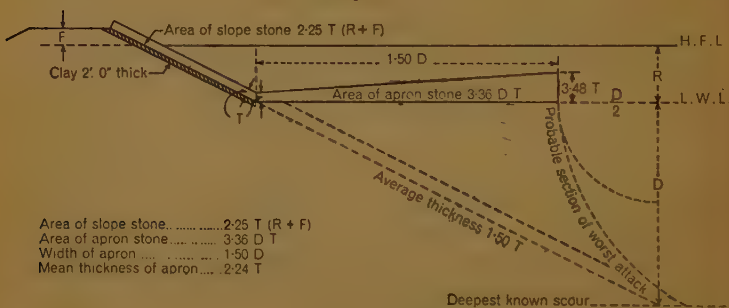
in its first flood season, the new Damukdia guide bank had not had very much of a test. *Fig. 2* also shows, however, that something abnormal occurred in the middle of September, for after falling a matter of 5·20 feet, the river started a late rise with great rapidity in the third week of September. The reason for this was the fact that there was, after the failure of the monsoon, a deluge of rain, lasting several days and causing severe floods, in the vicinity of Rohtak and Delhi in Northern India, and this flood water all found its way into the upper reaches of the Ganges. The result was an abnormal rise at the end of September, creating conditions at the bridge which the Author considers were without parallel or precedent. Heavy rain in Northern India frequently occurs and a rise in water level at the bridge is a natural result, but there was no reason to suspect that this particular rise would come down a distance of 1,000 miles and still retain a force which, in the light of subsequent events, proved to be the cause of the destruction of the right guide bank of the Hardinge bridge.

Investigations as to the cause of the breach have been exhaustive and, although various reasonable and probable theories have been suggested, the actual and definite cause of what happened on the 26th September, 1933, the day the damage was done, will probably remain an unsolved problem; there appears very little doubt, however, that the abnormal and sudden rise, after the river had fallen several feet, accompanied by a combination of the various circumstances and factors to which the breach has been attributed, were jointly or severally responsible for the breach.

Another possibility which the Author suggests, and which is a conclusion arrived at after close observation through the two subsequent flood seasons, is that the breach probably started, not by a slide into a hole made by an eddy at the toe, but high up on the slope of the bank, at or just below water level. Fast flow and specially fast-moving eddies visible on the surface, which have been discounted as harmless so long as they were moving, have appeared to the Author to create a very strong suction on the face of the boulder-protection of guide banks and aprons. If the covering of boulders is not sufficiently thick and the bank behind is of a material like silt or sand which almost "flows" in water, this material is sucked out through the boulders near and just below water level, rather than in great depths where the water is already very heavily laden with silt and sand and is carrying almost its maximum possible saturation. Water near the surface which is almost clear of silt and sand is more ready to suck out the covered bank and thus create a gap into which the boulders on the surface subside. Once this occurs, the earth or sand bank

above becomes exposed and, being fully saturated during the high-water season by seepage, proximity of the river, and rain, is very readily and easily removed by the flowing river. This process has only to go on sufficiently long, if unchecked, for the river to get sufficient elbow-room to increase the exposed area rapidly and create conditions which make a breach easy and inevitable. The Author has found in the last 2 years, by a careful watch on the guide banks, that this subsidence of the surface stone is what actually does occur, but further damage can be checked by promptly pitching boulders to cover the exposed face of the bank. If left for a few hours, however, all such subsidences become "slips," and these, judging by the speed at which they extend both longitudinally and inwards, would undoubtedly invariably develop into breaches. If these slips, which are first observed just below or just above

Fig. 3.



water-level, had worked up all the way from the toe of the boulder-slope, the area exposed would be so enormous that there could be no possibility of arresting their expansion by emergency pitching from the top, which at best is a very slow process.

The Author therefore recommends that there should be a revision in the design of guide banks in fast-flowing rivers or in rivers where there is much eddy flow. Experience has shown, firstly, that for a guide bank made of sand, a covering of 9 inches of clay under the boulder-facing on the slope is not sufficient; the clay should be not less than 2 feet thick and should be carefully laid without any gaps through which the sand behind can be sucked out. Secondly, it has shown that Spring's allowance [1] for stone in the apron, 2·82 DT, should be increased in all cases of fast flow to 3·36 DT, giving an apron with an average thickness of 1·50 T after descent.

Fig. 3 shows graphically the proposed design for a guide bank. By these two amendments in the design of guide banks built of sand,

[1] Footnote 1, p. 23.

the chances of subsidences, or slips, or, their last stage, breaches, will probably be considerably reduced.

The Breach.

There have been, ever since the completion of the bridge, watchmen at each of the protection banks, whose duties were to patrol it and to report any unusual occurrence to the officer in charge of the bridge. The watchmen's duties normally end at dusk, and on 25 September they reported, as on other days, that all was well. Judging by the speed with which slips develop, it is fair to assume that all was well that evening and that the damage really started late the same night or early on 26 September. A villager who usually proceeded to his work at 5 a.m., over the bridge, gave the first intimation to the stationmaster at Paksey station, stating rather vaguely that there was water flowing under the track on the right guide bank. This apparently was discussed in the station office, and the signaller stated that possibly something had occurred as he had heard in his cabin, at about 4 o'clock in the morning, a sound like the report of a gun from that direction. The stationmaster hurried to a point from where he could view the right guide bank, 1½ mile away, saw what looked like a breach, and was sending out an S O S telegram to all concerned, when the watchman whose duty was to patrol the guide bank appeared at his office and gave him definite information that the right guide bank had been breached. It was therefore about 6 o'clock in the morning before the officer in charge of the bridge heard the bad news. He promptly crossed the river in a motor boat to see whether the report was correct or exaggerated. Finding that it was only too true, he returned to the left bank to report the disaster to the Author, who immediately went across the river and saw a most terrifying sight: there was a breach in the guide bank 400 feet long, and this was increasing rapidly each minute in the direction of the bridge, whilst an embayment had formed behind the guide bank in which there was turbulent water from 50 to 60 feet deep; it was, in fact, dangerous to remain in this water in a light motor-boat while taking soundings. *Fig. 4* (p. 30) shows a few of the phases in the development of the breach.

There was neither labour nor stone at the site to make any serious attempt at preventing the breach extending, but fortunately there happened to be three assistant engineers in Paksey at the time, and these were immediately detailed to devote their attention to the organization of ballast trains for collecting boulders from the reserve depots, and to collect labour to start work. In the meantime all that could be done was to stand by and watch the terrifying spectacle

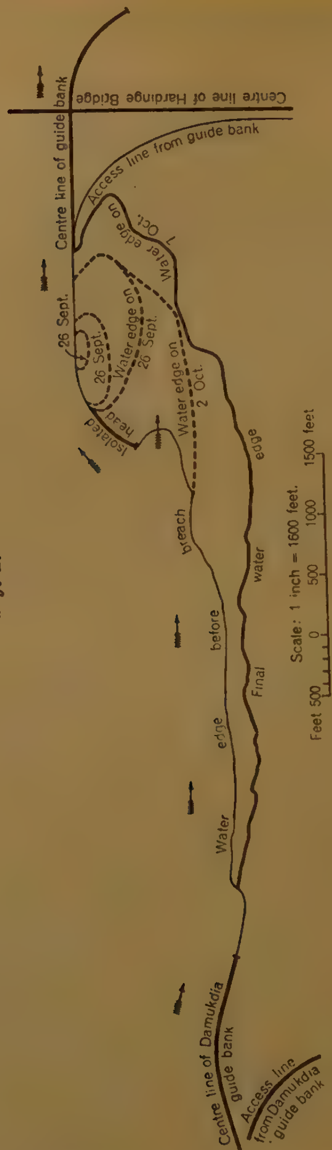


Fig. 4.

develop, and to release rail-lengths as the breach widened. The force of water was so strong that the 73-lb. double-headed track lying down the slope of the breach with its end in the water snapped off like a carrot with a loud report in two or three instances. Possibly

the report like a gun which was heard by the signaller at Paksey station was the sound of the first break in the rails when the track was suspended as a festoon right across the original breach. A few gangmen were collected and boulders were removed from elsewhere on the guide bank to throw into the breach in the hope that this might at least offer some resistance to its extension. This attempt being so inadequate and merely wasteful was abandoned, however, and nothing further could be done that day.

Observation of the stream while standing by showed that the river was not only turbulent, but came in spasmodic rushes at almost regular intervals. The Author has heard of the "breathing" of rivers, but he is doubtful whether any one, except those who were present at the site of this disaster on the 26th September, 1933, has ever witnessed "breathing" such as they saw. The river appeared to come into the embayment in rushes like exhalations, at intervals of about 2 minutes, creating a sudden afflux of about 2 feet. This wave rushed across the embayment, dashed against the eroding bank, expended its force there, and then, as if with each inhalation, carried away large tracts of land, covered with jungle and trees, in strips 10 to 20 feet wide and 30 to 50 feet long. This continued till mid-day, and then commenced to ease gradually until it stopped entirely at 4 p.m. The surface then became dead calm and there was no movement of the water in the embayment whatsoever; in fact it became almost a pool, except where the apron at the back of the northern end of the guide bank had been topped by the river and a stream was flowing in over this apron and out through the breach into the river. From these observations it appears as if the main force of the river, swollen by the floods in Upper India, had concentrated above the bridge in the form of almost a tidal wave or waves with a steep hydraulic gradient, which lasted over a period of about 12 hours. The following night passed without event and with practically no flow into the embayment through the breach, which during the day had reached a length of 1,000 feet, leaving an embayment behind 700 feet wide and 1,400 feet in length north to south. *Fig. 4* shows the limits of the breach at the end of the first day.

By the next morning material trains, loaded with boulders from the reserve depots, and a labour force arrived, and pitching into the breach and down both slopes of the guide bank was started and continued in shifts day and night. On the back slope an apron was thrown out, and on the front the existing original apron was strengthened by stacking additional stone on it. The river, however, continued to rise, and by the 2nd October the flow of water over the back apron on the isolated portion of the guide bank (referred

to hereafter as the "isolated head") had become so strong that it eroded the mainland and entirely isolated the head into an island. This unfortunately gave the river free flow into the embayment from the north and very soon widened the gap between the isolated head and the mainland to the extent of nearly 500 feet in a matter of a few hours, and eventually in the following days to 1,000 feet. It must be remembered that the whole countryside was flooded and that there was no access to any point of the attack except by water; the only craft at the bridge was a small 16-foot motor-boat and consequently nothing could possibly be done to arrest the expansion of this newly-forming gap at the north of the embayment. By the 3rd October a steamer arrived with two barges, and on the 5th and 6th October, as the river had commenced falling, an apron of boulders was thrown up on the mainland, opposite the isolated head, to arrest any development of that gap during the further fall of the river.

By the 5th October conditions appeared to have become quiet again, and discussions were immediately commenced as to the best method of repairing the damage. Had the fall of the river been gradual, nothing more might have occurred, but it is well known that when a river falls rapidly, considerable damage can be done to its banks and protection works. In fact, experience has shown that if the Ganges, at two-thirds high flood on the fall, should register a fall of even 12 inches in 24 hours, erosion of its banks is almost certain to occur. The Author thinks that this figure is sufficiently accurate to be considered as worthy of the attention of engineers employed on alluvial rivers, as he has found it correct on more than one occasion.

On the 7th October there was a fall of 14 inches, and with the high velocities resulting from the rush of water back into the main channels, a sudden attack developed just after dark. In spite of the mass of stone that had been pitched during the previous fortnight the breach extended in a few hours an additional 600 feet, thus making the total length eventually 1,600 feet. When this attack occurred, the stump of guide bank left by the breach, with the very considerable flow now coming on to it from the back of the isolated head, acted as a spur, and the usual result of this spur action, as is well known, is the creation of eddies and deep scour holes upstream and downstream of the spur. These newly-created eddies scoured so severely that pitching was of no avail, and even as far down as the bridge their effect was so severe that soundings taken at No. 2 pier at 6 p.m. and 8 p.m. showed that 40 feet of scour had occurred in this short interval. At the same time as this eddy was observed near the pier another eddy was observed about 200 feet down-

stream attacking the guide bank at chain 2 below the bridge, and it appeared as if there might soon be an additional breach of the guide bank at that point. Had a breach occurred here it would have immediately exposed the foundations of the land span approaches to the bridge, which are only 40 feet deep, and caused their destruction. Fortunately, with a large staff and labour now on the spot, pitching was commenced immediately at this point and carried on through the night, a second breach being averted. As, however, No. 2 pier had scoured to danger level, trains were piloted over the bridge for safety until daylight, when fresh soundings showed that no further scour had occurred and the restriction to traffic was removed.

The Scheme for the Safety of the Bridge.

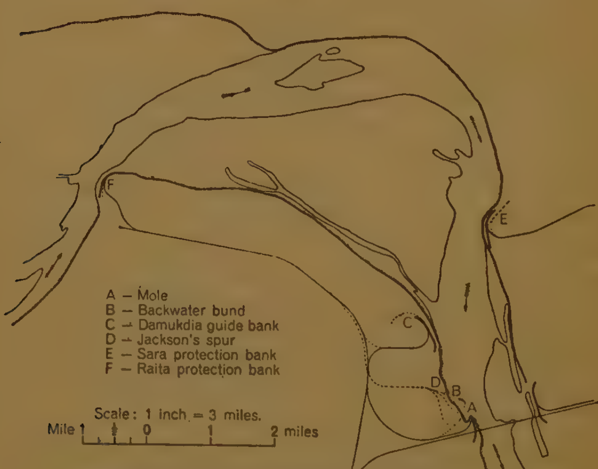
From the 8th October onwards, as the river fell gradually and there were no further changes in the conditions, a survey of the bed was taken in hand and a contour plan of the whole vicinity of the damage was prepared. This plan showed that a deep pot-hole had formed just below the stump of the guide bank, about 1,000 feet upstream of No. 2 pier, its depth being below the danger level of the pier. This, and the whole situation, showed that the bridge was left in a very vulnerable condition and, as such vast issues were involved, the Railway Board cabled to the High Commissioner for India requesting him to procure the services of Sir Robert Gales, M. Inst. C.E., to come to India and advise them as to what should be done.

It was known in advance that, no matter what Sir Robert Gales's scheme for the safety of the bridge might be, a vast quantity of boulders and a large labour force would be necessary, and a staff of engineers and inspectors of works would have to be appointed to carry out his scheme. The month of November was spent in these preliminaries and in such matters as housing, medical, and sanitary arrangements. All these arrangements were more or less ready when Sir Robert Gales arrived in Paksey on the 3rd December, 1933, and, after a searching preliminary inquiry, a scheme was drawn up by him (*Figs. 5* and *6*, pp. 34–5) for an immediate start to be made on the work, which he set forth in detail later in his report to the Railway Board. Sir Robert Gales not only dealt with the immediate emergency in his report, but included recommendations for works to be carried out against future developments and changes in the river. These further recommendations are not, however, detailed here, or embodied in *Figs. 5* and *6*, as, apart from the recommendation that the main breach should be closed, none of the further recommendations for other major works came into the period of construction covered by this Paper.

The work undertaken in the vicinity of the bridge was work that had to be completed before June, 1934, namely, within 6 months, and the ways and means adopted to get the work done within the limited period, the magnitude of the work itself, and the experience and knowledge gained in carrying it out, will form the subject-matter of the remainder of this Paper.

The possibility of closing the main breach itself was abandoned owing to the excessive depths of fast-flowing water over a length of 1,600 feet. The possibility of closing the northern gap between the mainland and the isolated head was also abandoned, as this, although not as deep as the main breach itself, had an average depth of 50 feet of flowing water, and even after closing it, there

Fig. 5.



SCHEME FOR THE PROTECTION OF THE BRIDGE.

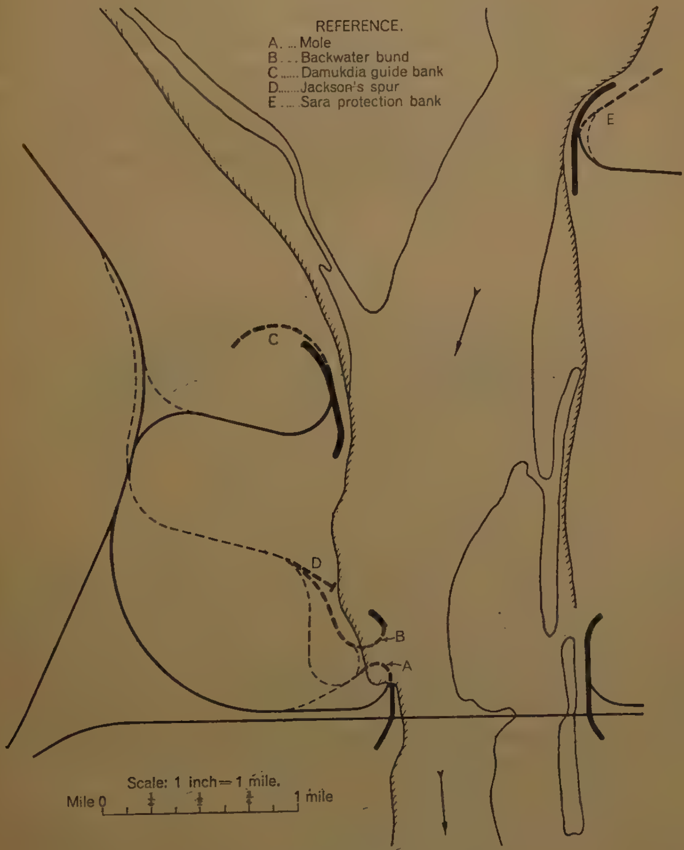
would still be the gaping breach below liable to extension by reverse flow into the breach and by the eddies which would be set up.

Sir Robert Gales considered that the immediate threat to the approach bank of the bridge at the back of the abutment should be removed by setting-out from the stump of the guide bank a stone-pitched head on a curve of 400 feet radius so as to provide a permanent work in the shallowest part of the embayment, which would afford some protection in case of damage to any further works which it might be found practicable to undertake further upstream. It was therefore decided that the first item of repair work should be the mole.

The Mole.—This is an embankment consisting of a stone dyke

built up from the bed of the river to slightly above low-water level, with an earth bank behind. The earthwork is prevented from spreading over the remainder of the embayment lying south of it (not because this is undesirable, but because it was impossible to fill the large lagoon that had been made there by the breach in the

Fig. 6.



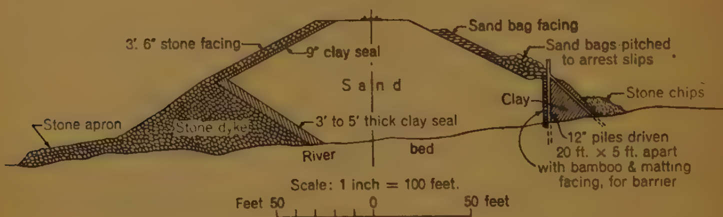
DETAILS OF WORK SHOWN IN FIG. 5.

time) by a temporary barrier made of piles driven 5 feet apart, faced down to the bed with a framework of treble matting and bamboos, and strutted behind with heavy timbering. Above low-water level the mole is an ordinary bank faced with a covering of boulders 3 feet 6 inches thick. The east end of the mole, at the stump of the breached guide bank, is in as much as 80 feet of water;

this decreases gradually to a minimum depth of 20 feet at the west end; the radius of the mole is 400 feet. A typical section of the mole is shown in *Fig. 7.*

The Backwater Bund.—Sir Robert Gales, after investigating various schemes for dealing with the top breach, including that of a submerged weir of boulders, decided upon what is considered to be the key to the success of the scheme carried out in 1934, and that was to construct what is called the backwater bund. This bund connects the downstream end of the "isolated head" with the mainland and may be considered the first step towards the restoration of the original guide bank. It is aligned across the shallowest part of the channel which had formed at the back of the guide bank. The construction of this bund eventually proved an extremely interesting piece of work because it involved the closing

Fig. 7.



TYPICAL SECTION OF MOLE.

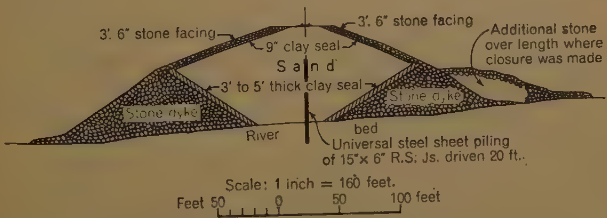
of what had become a part of the main channel of the Ganges carrying about $\frac{1}{5}$ of its total discharge. The backwater bund consists of two dykes of boulders built up from the bed of the river to slightly above low-water level, with earth filling between, and faced above low-water level, as in the mole, with boulders. As the inside slopes of the dykes would finish so close to each other, Sir Robert Gales decided that a definite seal against seepage through one dyke and thence easily through the narrow wedge of earth and out again through the second dyke was imperative. For this seal a line of steel sheet-piles was driven between the two dykes over a length of 800 feet where the dykes come nearest to each other at their base. Universal steel sheet-piling of 15-inch by 6-inch rolled steel joists was used, and driven an average of 20 feet into the bed. *Fig. 8* shows a typical section of the backwater bund.

CONSTRUCTION OF THE MOLE AND BACKWATER BUND.

A start was made on the mole towards the end of December, 1933, and was pushed ahead before the backwater bund was seriously

tackled, for at one time the Author was of the opinion that he had an almost impossible task, and that both these works could not, with certainty, be constructed within the time. Work was therefore concentrated on the mole to see what progress was possible, before work was started on the backwater bund. Weeks of toil showed almost no results, as the dissipation of stone in the deep flowing water at the east end of the mole was very considerable. The force of the current was, however, lessening daily with the fall of the river, and the dissipated stone travelled a lesser distance each day; eventually this dissipation almost disappeared and the base of the mole at its eastern end showed signs of coming into being. In the meantime, at the western end progress was gratifying as the dyke was in more or less still and shallow water. The construction of this dyke was pushed ahead, but a gap had to be left in its length wide enough for barges, loaded with sand, which was

Fig. 8.



TYPICAL SECTION OF BACKWATER BUND.

excavated from sandbanks out in the river, to be taken inside and unloaded. The interstices in the boulder dyke being considerable, it was decided that a seal of some kind down the back slope of the dyke was imperative if the sand were to be retained behind it. Work on the retirement of the protection bank at Sara (referred to later) had now commenced, and as a large quantity of black clay was available from there, it was transported to the mole in barges and dumped down the back face of the stone dyke. This Sara clay is a sticky, tough, black clay, almost completely impervious to water, and eminently suitable for such a seal. Ordinary earthwork, therefore, was confined to the southern end, up against the barrier, while the clay seal was being dumped down the back slope of the dyke. As it was impossible to ensure that this clay covered the whole face of the dyke, the original idea of a 6-inch to 9-inch seal was abandoned and clay was dumped to a thickness of as much as 5 feet, in the hope that any gap that there might be left in the dumping would eventually be filled by a part of the thick layer above sliding in to fill the gap.

The total length of the mole being about 1,100 feet, the number of barges which could be employed on boulder work, on earthwork through the gap, and on dumping the clay seal was naturally limited, and consequently the amount of work done in a day was limited. The Chief Engineer, Mr. E. B. Robey, decided that the only chance of completing the work in time was to work both day and night. A power-house was immediately erected and equipped with such machines as were available on the railway to provide floodlighting for night work ; the plant was ready by the 15th February, the labour force had been increased to 12,000 men, and night work was started.

Progress became so good that the work which had by now been begun on the backwater bund, and which was slow almost entirely on account of the output which could be loaded on to barges from the stone yard being greater than could be unloaded at the limited face of the mole, was immediately pushed ahead as fast as possible. The pitching of the two dykes of the backwater bund was commenced simultaneously, but the downstream one was kept in advance. The dykes were built by pitching boulders between a pair of floating bridges, made of old oil-drums and bamboos planked over and anchored in position on the alignment of these dykes, with a 5-foot gap between ensuring as accurate a curve of the dyke under water as was possible.

In building these dykes it was found that the boulders stood at a slope of about 1 to 1 under water. This was not, however, a stable angle of repose to carry a surcharge, and the dykes had been designed to slopes varying from 2½ to 1 in deep flowing water to 1½ to 1 in shallow still water. To make these slopes up to the required angle, rafts, made of 12-inch timbers framed into compartments 5 feet square, were moored in a regular pattern over the unfinished slopes and a calculated quantity of stone was dumped through each square in the raft sufficient to bring the slope below it up to full section. Daily sections were taken over this, as well as over all the work that was being done, so as to be up to date with the progress below water. The completion of these slopes was slow work and had to be done very carefully under strict supervision, as otherwise the mounds of stone that would accumulate as a result of indiscriminate dumping would set up serious troubles when the river was in flood. Cross sections taken when the work was completed showed that the slopes were almost exactly correct.

For expeditious earthwork between the dykes of the backwater bund it had been decided to build a timber-pile bridge so that trains of earth could be run on to this and be dumped quickly, but this scheme was abandoned as the earthwork done by barges in the preliminary stages was found to be quite fast enough and less

expensive. Every possible decked barge, flat and pontoon belonging to the Eastern Bengal railway, and which could be withdrawn from other work, was brought down to Paksey and used for boulder-work or earthwork, but soon the staff and labour had got so well into the routine of the job that a still larger output was possible; eleven 600-ton decked barges were therefore hired to increase the fleet. The fleet eventually consisted of four paddle steamers, two tugs, and twenty-six barges of an average capacity of 500 tons, and besides these, there were eight ballast-trains running to augment the earthwork and twenty-six rakes running as specials to bring in boulders from the quarries. Four old motor-boats were repaired and commissioned, and a new fast Thornycroft 140 horse-power motor-boat, equipped with an Admiralty type echo-sounding apparatus, was purchased; this sounding apparatus has been invaluable for taking soundings and making contour plans of the river-bed in the last two seasons. A novel addition to the methods of doing earthwork was the mobilization of a fleet of country boats of various sizes which were loaded from sandbanks, rowed or sailed to the site through the gaps in the dykes, and unloaded at the mole and backwater bund.

The daily output during the last half of March was so considerable that the stone dyke of the mole was above water throughout its length, except over the narrow shallow gap which had been left for country boats to enter, whilst earthwork appeared above water-level almost throughout the area by the 31st March. The earthwork above low-water level was speedily thrown up and finished with a layer of clay 9 inches to 1 foot thick before the 3-foot 6-inch boulder facing was placed, but at the east end of the mole the clay facing was made thicker and the boulder facing was increased to 5 feet. On the back slope of the mole, impounding a large area of water which had to be maintained at river-level by pumping so as to avoid a head of water accumulating, it was considered that turfing would suffice, but although this turfing took root rapidly, it proved useless for standing up to the seepage through the bank, and very early in the rise of the river the back slope commenced sliding into the lagoon. This was successfully arrested by pitching the bank with bags of sand where the slips were occurring, and as this first warning of weakness had come early it was decided to unload clay and the chips of stone and undersized boulders, which had accumulated on the work, behind the barrier to strengthen it, and to surface the whole of the turfed back slope of the mole with two layers of sandbags right up to high-flood level (*Fig.* 7, p. 36). This proved a successful way of arresting damage by seepage and is perhaps worthy of note for use in similar circumstances.

The closure of the backwater bund was perhaps the most spectacular of all the works that were done, as the stream above was about 1,000 feet wide, and a discharge of 50,000 cusecs was flowing with a velocity of over 2 feet per second. To accomplish this closure the downstream dyke was built from both ends, increasing in width as the gap reduced. The last 80 feet were finished 5 feet below water level, but widened even still further over this length, forming almost a weir (*Fig. 8*). The velocity by this restriction was considerably increased, and became still greater as the clay seal was dumped behind the dyke. When the seal was completed and the stream was restricted to only an 80-foot gap 5 feet deep the main river was nearly at its lowest velocity, about 1 foot per second; the velocity over the weir was, however, about 2 feet per second and to prevent retrogression a small apron was thrown out at the downstream toe of the weir. It was unwise to wait for a further reduction in the velocity of the river, as it was now the middle of March and much remained to be done, and all available barges loaded with stone were therefore brought to the gap, and pitching was done continuously for about 8 hours, as fast as possible, until the gap was raised above water level. In accomplishing this a large quantity of stone was dissipated, and carried along the weir, but all the same the quick and continuous pitching closed the gap, leaving merely the interstices between the boulders as a safety valve against the water pressure. The width of boulders above water was then temporarily widened and the clay seal dumped behind it without any trouble. This success gave the staff great confidence in themselves and greater zest to complete the work that remained.

Over the eastern half of the embayment left between the backwater bund and the mole a carpet of boulders 2 feet thick was laid to prevent the eddies, which were bound to form in this sheltered water, from deepening the bed and endangering the dykes of stone. The quantities of stone boulders and earthwork put into the mole and backwater bund worked out to 1·32 *crores* (13,200,000) cubic feet of boulders, and 4·41 *crores* (44,100,000) cubic feet of earthwork.

Other Works Carried Out.

Jackson's Spur.—To control the width of the gap between the "isolated head" and the mainland, and to direct the flow if the river should cut in below the Damukdia guide bank, Sir Robert Gales recommended that the temporary apron which was thrown out during the emergencies in the previous October should be completed in the form of a protection bank, with an access line running westwards to join with the Damukdia branch of the Eastern Bengal

railway. This spur is 1,600 feet long and for a length of 1,000 feet has an apron of 1,100 cubic feet of stone per foot run; it is aligned 175 feet upstream of the tangent to the curved isolated head (*Fig. 6*, p. 35).

Marginal Banks.—To hold up the spill from the river between the three major works, a marginal bank was constructed connecting Jackson's spur with the backwater bund, and thence along to the western end of the mole. This was protected with boulder facing, and was given a shallow apron varying from 100 to 400 cubic feet of stone per foot run to prevent damage to the slopes. The completion of the marginal banks and access lines to the mole and backwater bund were the last works done in the short season, and were finished only just ahead of the rise of the river, in fact after the rise had commenced.

Damukdia.—Sir Robert Gales also recommended that the upstream part of the Damukdia guide bank, which had not been fully stone-pitched in 1933, should be slightly re-aligned to reduce the possibility of deflection of the main stream towards the left bank when the river re-occupied the Damukdia channel, and should be extended to reduce the possibility of spill passing to the back of the guide bank, whilst the stone pitching should be made up to the full amount recommended for this guide bank. This work was done.

Sara.—At Sara, Sir Robert Gales recommended the cutting-down of the remaining part of the projecting spur to low water and below by hollowing out as before, the setting-out of the siding at 60 degrees, and the laying of 1,800 cubic feet of boulder apron for a further extension of 500 feet to deal with the erosion anticipated during the next floods. He also recommended that the slips between the sunken head and the extension should be filled with stone and the alignment of this stone-pitched river bank maintained. It was understood that the curving-back of the Sara protection bank with the 60-degree tangent extension was a temporary measure designed for the purpose of putting a stop to the gigantic eddies which had deflected the river, and of preventing the protection bank from being outflanked and attacked again as a spur, and that as soon as the restoration of the right guide bank had been completed, advantage would be taken of any favourable opportunity for constructing some permanent work which would limit the erosion eastward of the Lalpur bight and further improve the direction of the flow of water towards the bridge.

Raita.—At Raita, as there were indications of an embayment developing similar to that which had occurred at Sara, it was decided to take timely action and to extend the head of this protection bank before the embayment had time to set up dangerous conditions. This was a small work in comparison with the others

that were done during the year, but nevertheless was an additional drain on the limited stone supply and on the energies of the staff.

A description and plan of the training works as originally constructed will be found in Sir Robert Gales's Paper.[1] The present state of development of the training works, including the added Damukdia guide bank, the modification of the Sara protection bank, and the right guide bank before the closing of the gap, are shown in *Fig. 5* (p. 34) of the present Paper.

Borings.—Another of the recommendations embodied in Sir Robert Gales's report was that borings should be put down around the piers to discover whether the large quantities of pitching that had been placed there when the bridge was built, and the stone which had been added since, had been really washed away as completely as had been suggested, or had only sunk to deeper levels round the piers when scour had occurred. The method adopted in the past for finding whether the stone pitching round piers or on aprons had fallen where it was wanted, or whether it had been washed away, was by "pricking." This consists of jumping a ⅞-inch hexagonal steel rod vertically through guides by slinging it over a pulley, power being supplied by a gang of coolies. Although cheap and effectual through from 10 to 16 feet of silt, it is ineffectual through greater deposits, as the rod invariably jams and prickings have to be abandoned before positive information is obtained. Boring, being obviously a more positive method, was adopted and will be adopted in future in the annual investigations which are carried out to locate any pitching at greater depths than below 15 feet of silt, or through greater depths than 50 feet of water, where prickings are not very satisfactory.

A series of 4-inch borings was accordingly put down and showed at what level the stone existed, if it were there at all. In most cases it was proved that pitching placed around piers at a reasonable depth is not wasted, but sinks almost vertically as scour occurs; some stone on the other hand is washed away and some is carried downstream of the piers to a distance where it is too far to be helpful in supporting the pier. On the results of these borings it was decided that the piers exposed to special attack by scour, due to the existence of the clay patch described hereafter, or to danger by the extension of the deep depression in the river bed 1,000 feet upstream of the bridge, should be pitched heavily, not any longer with "one-man-rock" but with boulders weighing from 1,000 to 2,000 lbs. each.

Special quarries had to be opened to obtain such boulders and

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[1] "The Hardinge Bridge over the Lower Ganges at Sara." Minutes of Proceedings Inst. C.E., vol. ccv (1917-1918, Part I), p. 18.

special arrangements had to be made for skidding them down the quarry faces for loading into wagons. The first consignment arrived at the bridge late in February and the pitching was commenced at once. At first a crane was used for unloading the boulders on to barges, one by one, and then unloading them by crane from the barges round the piers; this proved too slow a method and was abandoned in favour of man-hauling the boulders by ropes out of the wagons, and down skids of greased rails, on to large square pontoons. By this method several wagons were dealt with simultaneously and the loaded pontoons were then positioned round the piers, where the boulders were unloaded in a pattern, exactly where they were wanted, by gangs of men tipping them over the edge. The work was laborious and slow, as a total of 15,000 tons of these large boulders had to be deposited in a pattern. This consequently meant day and night work by flood-lighting and was only completed on the 12th June, just one day before the main flood started.

Clay.—In addition to the borings for locating the pitching, borings were put down along the centre line of the bridge to locate a clay patch which was struck at the time of the construction of the bridge; it was felt that the existence of this clay, resisting the scour of the bed between piers Nos. 3, 4, 5, and 6, where it was known to have existed, might be responsible for the deepening at the adjoining piers, or, by a weir action, for creating the dangerous large holes which were charted downstream of the bridge.

These borings were extended over a large area upstream of the bridge and over a smaller area downstream and they exposed the existence of this clay patch most markedly. It was found to exist between Nos. 4 and 5 piers at about R.L. 175·00 and to extend about 1,000 feet north-eastwards, from No. 5 pier, but dipping to R.L. 150·00; the clay round Nos. 3 and 6 piers was also located, but was scoured away in the floods of 1935.

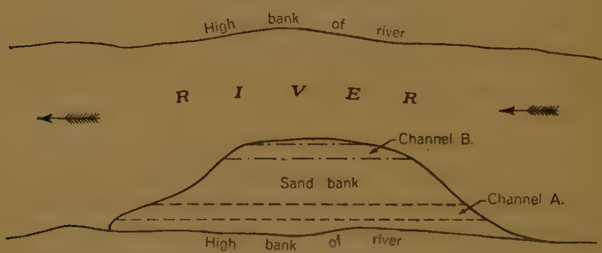
Cutting Channels, Ploughing Sandbanks.—At the end of the work, and before the river had risen sufficiently to cover the sandbanks in the vicinity of the bridge, the Author decided to try an experiment to assist the river to remove its activities from the right bank to nearer midstream, or at least to divert some of its energies from the right bank. For this a channel 100 feet wide and varying in depth to a common R.L. of 225·00, was excavated across the sandbank on a line which appeared a likely direction for the stream, or a part of it, to take. The Author wishes to record the futility of this experiment or of any attempt to provide a river like the Ganges with new channels by "scratching" excavations. It has, however, been found subsequently that if channels are cut in a sand bank on the edge of the stream, they certainly prove efficacious in widening

the main channel. *Fig. 9* shows channel A, which was the futile attempt, and also channel B, which definitely proved useful.

Medical Arrangements.

On the 9th April, 1934, a case of cholera occurred in the Hardinge bridge camps in spite of as perfect preventive arrangements as were possible in temporary camps. In 3 days there were twelve deaths, and two thousand coolies absconded in spite of every effort, and police support, to detain them; on the sixth day two thousand more men absconded. It looked as if the work was going to be left unfinished and as if the bridge could not be saved; but on the ninth

Fig. 9.



Channels Cut in Sandbank.

day, after a total casualty list of seventy-four deaths, the epidemic was blotted out by the Chief Medical Officer, Dr. C. D. Newman, and an augmented staff of doctors and inoculators working day and night. The warning from this is that in addition to satisfactory and even expensive medical and sanitary arrangements, which are never a waste of money, it is advisable to inoculate labour before it comes into the area of work, for the first cases were traced down to a new gang of coolies who arrived from an infected village on the 7th April.

Cost of Work.

Details of the cost of all work done in the years 1933, 1934, and 1935, are given in the Appendix.

Flood Season, 1934.

All works were completed, some only just ahead of the rise of the river, and although there was a general belief that the bridge was now quite safe there was no certainty that the new protection works and the scheme would stand up to the flood, and that all danger to the bridge had been removed. Accordingly every known precaution was taken to preserve the guide banks and the bridge through the flood. Chosen inspectors and assistants were detailed

for a day and night patrol of 1,000 feet of guide bank each in shifts of 4 hours on and 8 hours off. By night the banks were lit by flood-lights to make any movement of stone easily detectable, and pier soundings were taken by special staff, in shifts, night and day. An officer was posted on night duty at the site and an elaborate system of alarm signals was introduced so that at the first sign of trouble or damage action could be taken. A nucleus of six hundred coolies were paid a daily wage with nothing to do except to be available night or day, if required.

The floods started more or less to schedule about the middle of June, with a rapid and continuous rise of 10 feet by the middle of July. Then came a slight fall, followed again by a rise, and by the middle of August normal high-flood level was reached. The river then commenced to fall gradually till the end of August, when floods occurred in Bihar. This was a warning for the Hardinge bridge, and when eventually, on the 3rd September, the river at the bridge touched the highest level it had reached since the days of its construction, with a discharge of 1,750,000 cusecs and a surface velocity of 13·2 feet per second, everything was in readiness to arrest the first slip, which occurred that afternoon at the upper end of the "isolated head"; emergency pitching involving the use of 7 *lakhs* (700,000) cubic feet of boulders saved the head from demolition. The flood then commenced to fall from this peak, and by the middle of October danger of the piers scouring had passed. It appeared as if further damage to the guide bank was unlikely.

Once again the unexpected happened, and on the 16th October a sudden attack, following a fall of only 9 inches, resulted in a very serious slip at the south-east corner of the junction between the "isolated head" and the backwater bund, extending so far into the bank that it seemed impossible to save a breach and to prevent the demolition of the old "isolated head"; even though this was 2,000 feet upstream of the bridge, it might easily have created very dangerous eddies and deep scour at the piers, but the extension of the slip was arrested before it cut through to the backwater, and once again all was safe. Another slip occurred 10 days later at chain 10 of the guide bank, and extended down as far as chain 5.50. This, although nearer to the bridge, was not so serious, even though 6 *lakhs* (600,000) cubic feet of boulders had to be pitched before it was repaired. Nothing more occurred, and the river continued a gradual and regular fall to its lowest water level in the following April.

Restoration of the Breach.

The Railway Board had in the meantime decided that, in addition to having the advantage of the recommendations made by Sir Robert Gales, for the further works necessary they would seek the

opinion of a committee of engineers in India under the chairmanship of Sir James Williamson, Agent of the Bengal and North Western Railway, who had had considerable service and experience on river-training work in the watershed of the Upper Ganges. This Committee met at Paksey, both during and again immediately after the floods of 1934, and their final recommendations were submitted to the Railway Board of India in January, 1936. They confirmed the recommendation made by Sir Robert Gales that the main breach in the right guide bank should be closed as early as possible, but considered that, before further works were undertaken, experiments should be carried out in the Irrigation Department's experimental laboratory at Poona (with the permission of the Government of Bombay), by Mr. C. C. Inglis, C.I.E., B.A., B.A.I., M. Inst. C.E., with a working model of the river Ganges, to see whether any of a series of proposals, with which they supplemented Sir Robert Gales's recommendations, would afford a scheme of permanent safety for the bridge. The results of these experiments when published by Mr. Inglis will be of the very greatest interest, but as they form no part of the work done in the low-water seasons of 1934 and 1935, they do not enter into the scope of this Paper.

The main work in the season 1935 was therefore to carry out the recommendation of Sir Robert Gales, which was confirmed by the committee of engineers, namely, the restoration of the guide bank across the original breach. This restoration is of the same design as the mole, being a dyke of stone, sealed behind with clay and backed with earth, which is held from spreading by a pile-and-matting barrier. The quantities were, however, considerably greater than in the mole, for the bank was 1,600 feet long and the average depth of water at low level over the length was 65 feet. To visualize this bank of earth and stone, it has, at its maximum section, a base 150 yards wide and a height of 105 feet, and this decreases only very slightly over the greater part of its length.

The same methods of construction were employed as in the case of the mole, but there was an incident during the construction which appears sufficiently interesting to be recorded here. Instead of erecting the pile barrier at the toe of the bank, where the back slope reached the bed, it was fixed at a distance varying from 100 feet to 300 feet away, both to obtain a shallower alignment for piling in not more than 40 feet of water, and to provide a berm to the bank at low-water level; this was considered advisable even although it increased the quantity of earthwork. Clay was not available in large quantities as in the previous year from the work at Sara, and it was considered that if a seal on the back slope of the dyke were provided down to a level where the dyke was 100 feet wide it would be sufficient, as sand was not likely to flow through

the interstices in so great a width of boulders. Earthwork proceeded satisfactorily, and daily sections showed that the bank was gradually rising under water, until one day the soundings showed that the whole area had gone down by as much as 20 feet. This was early in April and was a most alarming state of affairs; the only explanation possible was that sand did flow through the interstices in even a greater width than 100 feet of boulders. The time left for completion was so short that once again drastic measures were necessary. The idea of the berm from 100 to 300 feet wide at the bank and the pile barrier were abandoned, and, instead, a pyramid of sandbags was built on the line where the toe of the bank reached the bed, thus reducing very considerably the quantity of earthwork to be done in the time. To prevent a recurrence of the slide of sand through the boulders, all earthwork was stopped and the labour and barges were diverted to bringing clay, purchased from villages 3 and 4 miles upstream, to seal the whole of the back slope of the dyke. When this was completed, or at least done as satisfactorily as time and the conditions permitted, earthwork was started again and completed to water-level over the reduced area without further difficulty. The sandbag pyramid was then backed with stone chips and undersized boulders, and the main bank was thrown up at the rate of 3 *lakhs* (300,000) cubic feet per day, backed with two layers of sandbags and faced with clay and boulders. The bank was completed and the track linked thereon by the 1st June. This work, the repairs to the previous flood-season's slips, and other subsidiary works carried out between December, 1934, and June, 1935, cost approximately £300,000.

Flood Season, 1935.

The floods of 1935 reached the highest level for the year in the third week of August and were only 6 inches below the previous year's abnormal high-flood level. The highest velocity recorded in this flood was 13·6 feet per second with a discharge of 1,500,000 cusecs. Repercussions of the flood season remain a source of danger and damage for a month after the last fall commences, in fact until the rate of fall becomes less than the safe maximum of, say, 4 inches a day, but in 1935 nothing untoward occurred. Slips and scour occurred during the month of August and were dealt with. Piers Nos. 5, 6, and 7 were all scoured deep, and although not to the danger limit they showed a tendency towards it; in each instance emergency pitching (although only with "one-man-rock"), was done from barges which were towed through the turbulent water and eddies below the bridge and moored at the piers.

This used to be slow work, and was rather a forlorn attempt of pitching stone into water and hoping it would reach the spot for

which it was intended. Practice and experiments made with "one-man-rocks" tied to piano wire and pitched in water flowing at a velocity of about 12 feet per second showed that boulders travelled to the bed at an angle of about 45 degrees, and this calculation has been varied and adopted with success during two seasons for pitching around piers in fast-flowing water.

The experience gained during the flood season of 1935 shows that when a slip is detected (and especially if it is detected early and is probably over a length of no more than from 20 to 50 feet), besides the pitching which is normally done on the exposed face of earth to cover it and to break the contact between it and the flowing water, it is essential to prevent lateral extension. The most efficacious method of arresting slips and preventing their rapid extension was found to be to pitch boulders speedily over a length of about 15 feet beyond the slip, at both ends, at water level, for it is at these points that slips extend, and a mass of stone speedily deposited here prevents, or at least slows up, not only their extension but the depth to which they erode into the bank. This added stone feeds the action of the slip and preserves the original slope, and it does not therefore remain to form a dangerous protuberance on the bank. *Figs. 10* show how this method was successfully adopted on two occasions, one when dealing with a single slip and the other when two slips occurred on consecutive days. It also shows how the damage in the latter case was confined to two small loops rather than their combination into one large ugly loop.

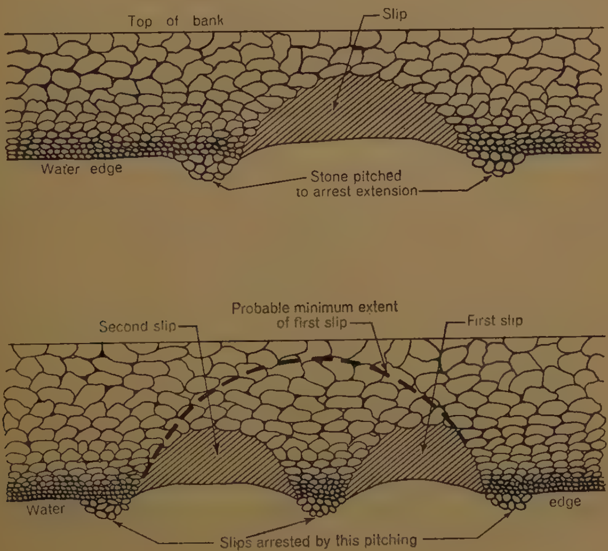
Another experience gained in dealing with slips and worthy of mentioning is that if a slip is feared for any reason, such as a marked settlement of the guide bank or a violent eddy suction persisting along a particular length, a warning of trouble might be obtained by throwing say four or five pieces of "one-man-rock" simultaneously on to the suspected slope as near the water-edge and as close together as practicable, and noticing at the moment of impact whether there is a general looseness of the boulders on the slope. This warning enables the watch at this point to be more careful and labour to be handy to start pitching immediately a slip actually shows signs of starting. It is not advisable to pitch in anticipation of a slip occurring for, should the slip not occur, the boulders pitched in anticipation merely form a projection on the guide-bank slope and result in disturbances in the flow, and invariably set up trouble.

Conclusion.

The condition of the river towards the end of the flood season of 1935, the waywardness of its channels for several miles above the

bridge, and the daily-varying course of the main current near the bridge itself, present a most interesting study. Channels above Raita appear to be moving rapidly into new, or rather into old, alignments which, should they develop, will change the whole regime of the river, because the position of these upstream channels will influence the alignment of the main stream at the bridge, where the present indications are for it to move away eastwards. In fact, the river appears to be in a state of unstable equilibrium and might rapidly assume an alignment through the bridge which will solve

Figs. 10.



Slips on Guide-Banks.

the problems of those who remain to look after it. This statement is intentionally (and advisedly) made in an indefinite and guarded manner, because the Author is fully alive to the fact that it is impossible to be prophetic in dealing with the problems of the river Ganges and of rivers in the alluvial plains of Bengal.

Acknowledgements.

The Author acknowledges the great use he has made of the information in Sir Francis Spring's Paper,[1] and the frequent reference which he has made to Sir Robert Gales's Paper on "The Hardinge Bridge over the Lower Ganges at Sara." [2] Both these Papers have been invaluable to him and his staff.

[1] Footnote 1, p. 23.

[2] Footnote 1, p. 42.

During the two years since the breach occurred the Author has been fortunate to have, in turn, as his Chief Engineer, Mr. L. F. Jackson, Mr. E. B. Robey, Mr. A. F. Harvey, F.C.H., and Mr. F. R. Morgan, M. Inst. C.E., who have instructed and guided him and his staff in their work, and who are here thanked for the permission granted to him to write this Paper.

Staff.

The Author has been in charge of the Hardinge bridge continuously from the construction of the Damukdia guide bank until the conclusion of the 1935 flood season, having as his executive engineers in turn, Mr. K. B. Roy, Lt.-Col. E. F. Johnston, and Mr. H. K. Koregoakar, M.A., B.Sc., Assoc. M. Inst. C.E., whom he takes this opportunity of thanking for their work; he also thanks his assistant engineers, Messrs. H. V. Langley, H. R. von Lintzgy, and S. N. Sen Gupta, who worked indefatigably on the constructions during the last 2 years; Mr. S. K. Bhattacharjee who started up the boulder quarries; Mr. M. A. Bary who operated them so successfully; and Mr. J. L. Puri, assistant traffic officer, in charge of the transportation of the boulder trains.

APPENDIX.

Cost.

Boulder work, inclusive, cost an average of Rs. 20/–, or £1 10*s*. 0*d*., per 100 cubic feet, and earthwork Rs. 16/– or £1 4*s*. 0*d*., per 1,000 cubic feet.

The following are the costs of the various works done in 1933, 1934 and 1935.

		£
1933:	Damukdia guide bank and other lesser works	150,000
1934:	Mole	143,600
	Backwater bund	102,200
	Damukdia	14,500
	Sara	26,300
	Raita	5,000
	Pitching piers with heavy boulders	17,100
	Jackson's spur, marginal banks, etc.	50,000
	Miscellaneous expenditure, including power-house, etc.	55,500
	Plant and general charges	35,200
	Emergency pitching during the flood season	30,800
1935:	Restoration of right guide bank and subsidiary works	300,000
	Total	£930,200

Paper No. 5034.

"Newry Ship-Canal Improvement Scheme."

By Robert Ferguson, B.A., B.E., M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*) [1]

TABLE OF CONTENTS.

Historical.

In a Paper [2] by Sir John Rennie, Past-President Inst. C.E., in 1856, a description was given of the natural navigation of Newry port prior to the construction of the ship-canal, basin, and inland canal, which was commenced in 1761 under the authority of the Board of Inland Navigation. An account of the works carried out by Sir John Rennie between 1842 and 1850 for the improvement of navigation in the lower division or estuary of the river Newry, and the extension and enlargement of the ship-canal, is also contained in the same Paper.

The ship-canal was extended, under Sir John Rennie's scheme, for a distance of 1½ mile downstream from the old lock at Fathom to the new entrance-lock (now known as Victoria lock) which was constructed at the same time, whilst the old canal from Fathom to the basin at Newry was deepened. The basin was also enlarged at that time and is now known as Albert basin (*Fig. 1*, p. 52). The tidal navigation between Victoria lock and Carlingford lough was deepened, straightened and otherwise improved so as to correspond with the dimensions of the new canal. The new canal from Fathom to Victoria lock was constructed with a bottom width of 70 feet, a depth of water

[1] Correspondence on this Paper can be accepted until the 15th March, 1937, and will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

[2] "On the Improvement of the Navigation of the River Newry," Minutes of Proceedings Inst. C.E., vol. x (1850–1851), p. 277.

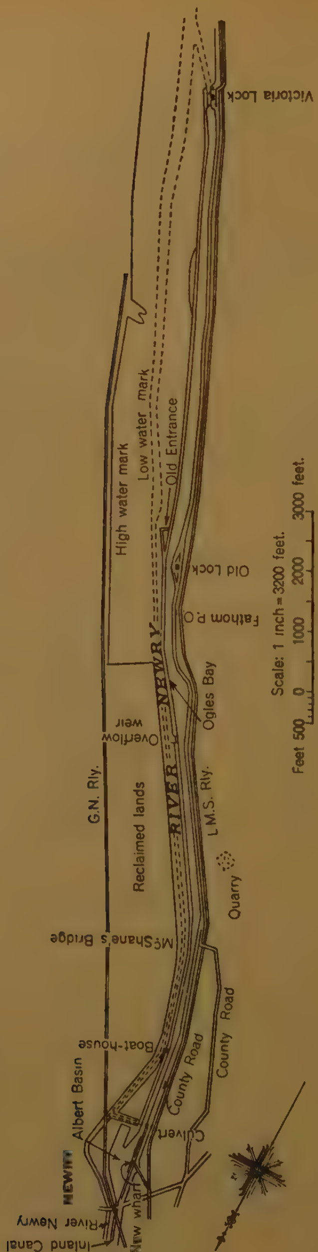


Fig. 1.

General Plan.

of 15 feet 6 inches, underwater side-slopes of 2 to 1, and a width at the surface of 160 feet. The old canal, which continued to form part of the waterway to Newry, was enlarged as far as the existing banks and property of the Navigation Company would permit.

At an early stage the restricted channel of the old canal prevented full advantage being taken of the facilities for navigation that had been provided by the formation of the new channel in the tidal waters, the construction of the new portion of the canal, and the enlargement of Albert basin; soon after 1850, therefore, the water-level in the canal was raised by about 1 foot in order to improve navigation in the old canal between Fathom and Newry.

Nevertheless, the old canal, which had an effective depth of water of only 10 feet over a width of fairway of 30 feet, continued to cramp the navigation of the port, and with the passage of time conditions became almost intolerable. The navigation was unsuited to standard vessels of modern design, and shipowners making frequent use of the port were compelled to build steamers with a mid-ship section that was specially designed to suit the profile of the waterway on this portion of the canal. Moreover, the wash from steamers passing through the restricted channel of the old canal caused excessive wear on the pitched slopes and banks, which required constant renewals and attention, and the annual expenditure in respect of bank repair-work became an increasing drain on the finances of the Harbour Trust.

Ship Canal and Tidal River.

The masonry culvert, constructed in 1850 for the purpose of conveying the waters of the Derrybeg river under the ship canal at the southern end of the Albert basin, collapsed in October, 1928, and permitted the waters of the canal to escape into the tidal channel of the river Newry. The impounded water-level in the ship-canal is about 3 feet 6 inches above H.W.O.S.T., and following the collapse of the culvert the tide flowed and ebbed in the canal, so that all navigation was brought to a standstill. The sudden release of the impounded water and the rapid fall in the water-level brought about the collapse of a portion of the quay-wall on the east side of Albert basin, and caused a number of slips on the banks of the ship-canal.

Owing to the nature of the substrata encountered in the excavations for the foundations, the reconstruction of the culvert proved tedious, and the fact that the new culvert had to be founded at a lower level than the old one in order to provide for future developments increased the difficulties of reconstruction. The port was not reopened to traffic until 8 months after the collapse of the culvert,

and during this period of shipping inactivity the public of Newry had ample time to realize the important part played by the Port of Newry in the trade and commerce of the town.

Consequently, the attention of the citizens was focussed upon the undertaking of the port, and the traders urged the provision of better shipping-facilities. Public opinion was divided, however, as to the best method of improving the navigation. One section favoured the improvement of the ship-canal whilst the other section was of opinion that the natural navigation by way of the river Newry should be reverted to, as it offered greater scope for future development. Full reports on each scheme, together with detailed estimates, were prepared by the Author's firm. These reports were submitted to a public meeting which was convened for the purpose of ascertaining the views of the ratepayers on the matter.

The estimated cost of the tidal-river scheme was £146,000, and the scheme involved the dredging of 600,000 cubic yards of material in the bed of the river and a considerable quantity of stone-pitching on the slopes of the proposed channel, as well as the construction of a new entrance-lock into Albert basin. The estimated cost of maintenance dredging was £2,000 per annum. Eminent engineers in the past, including Sir John Rennie, had consistently reported against the adoption of a navigation scheme in the narrow upper reaches of the river Newry in place of the ship-canal, on account of the large amount of dredging involved in the formation of the channel and the heavy annual maintenance-dredging that would be necessary to remove the large quantity of mud that rapidly accumulates in the estuary. Moreover, the Government of Northern Ireland was prepared to give a grant from public funds for the improvement of navigation in the ship-canal.

The public meeting of the ratepayers of Newry decided in favour of the scheme for the improvement of the ship-canal.

Introduction.

Owing to the defects referred to, the ship-canal did not admit vessels large enough for the economical carriage of goods under modern conditions, and statutory powers were obtained in 1929 to enable the Newry Port and Harbour Trust to execute works for the improvement of the port.

At the outset the Harbour Trust instructed the Consulting Engineers to prepare a scheme for the improvement of the ship-canal so as to provide navigation-facilities for vessels of 1,000 tons gross. After consultation with several shipping and shipbuilding authorities it was concluded that a vessel of this tonnage might attain the

following maximum measurements in any one of its three principal dimensions, namely :—

Length, over all . .	240 feet, with a tendency to decrease in favour of greater beam and draught.
Beam ,, . .	34 feet with a tendency to increase to 35 feet.
Draught ,, . .	15 feet with a tendency to increase to 15½ feet.

The navigation-facilities for this size of vessel would have necessitated the extension of the Victoria lock by about 50 feet and the deepening of the ship-canal to provide a depth of at least 16½ feet of water over a bottom width of 50 feet. The cost of executing the works necessary for the navigation of vessels of 1,000 tons gross was estimated at £80,000. In anticipation of this scheme being adopted, the foundations of the culvert, when being reconstructed in 1929 under the supervision of Mr. T. S. Gilbert, M. Inst. C.E., were fixed at such a level as to provide a depth of water over the top of the culvert of 16½ feet.

As the trade of the port consists mainly of the import of coal, timber and grain and the export of cattle, and as such traffic is carried usually in coastal vessels of from 400 to 600 tons gross, it was felt that the present-day requirements of the port would be adequately served by providing facilities for a smaller vessel than one having a gross tonnage of 1,000 tons. Accordingly, the Trust instructed the Consulting Engineers to prepare a modified scheme that would enable vessels of 500 tons gross to use the port, but the works were to be designed in such a manner that, merely by subsequent dredging, navigation-facilities could be provided in the future for vessels up to 800 tons gross. The modified improvement scheme forms the subject of this Paper.

The sectional area of the water-way at the narrowest places on the old canal was approximately 600 square feet, whereas the immersed sectional area of a vessel at midship of 500 tons gross when loaded is approximately 350 square feet. In order to reduce wear and tear on the banks as far as possible it was essential that the ratio of sectional area of waterway to immersed midship sectional area should be as large as practicable, and in any case should not be less than 3 to 1.

Thus, the minimum sectional area of the improved waterway would have to be 1,050 square feet. The draught of a vessel of 500 tons gross is, when loaded, about 12 feet 6 inches, and it is desirable to have a clearance of 2 feet of water under the keel; the depth of water to be provided was thus determined at 14 feet 6 inches. After investigation on the site it was decided that the unprotected underwater slopes, below a depth of 3 feet from the surface, would

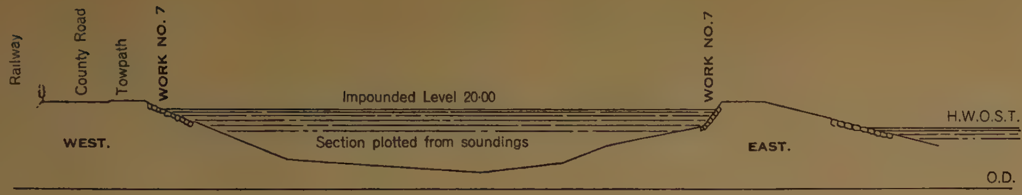
be stable if dressed at a slope of 1 in $1\frac{1}{2}$. A berm 3 feet in width at a depth of 3 feet below the surface was considered desirable at the top of each slope. For reasons that will be referred to later, it was decided to effect the necessary widening on the east bank, and after a careful survey it was found that a minimum width of 92 feet at the surface could be obtained without encroaching unduly on the waterway of the river Newry. Accordingly, the bottom width of the ship-canal when improved was determined at 51 feet 6 inches, and a waterway having a minimum sectional area of 1,066 square feet was thus ensured. The sectional area of the waterway in the new canal from Victoria lock to Fathom (*Fig. 2*) already exceeds 1,066 square feet, so that the improvement works on the waterway were confined almost exclusively to the old canal from Fathom to Albert basin.

The deepening of the waterway in the future to a depth of 15 feet 6 inches over a bottom width of 48 feet 6 inches will provide navigation-facilities for a vessel of 800 tons gross, but in order that it may pass through Victoria lock the length of any such vessel must not exceed 198 feet, which is the effective length of the present entrance-lock chamber.

Reference has been made already to the fact that the old canal between Fathom and Newry had been enlarged and deepened as far as the banks and property would permit. The first step in the design for the improvement of the waterway was to determine how the necessary increase, varying from 12 feet to 27 feet, in the surface-width of the old canal was to be obtained. The only ground on the west bank that was available for widening purposes was the original tow-path, having a width of about 9 feet; this was inadequate and its removal would have deprived the road, which ran beyond it, of a measure of protection, for which some substitute would have to be provided. Moreover, the narrow width of the tow-path did not permit of the provision of a suitable anchorage for tying back a revetment of sheet-piling, which was a second method by which an increase in the waterway might be effected.

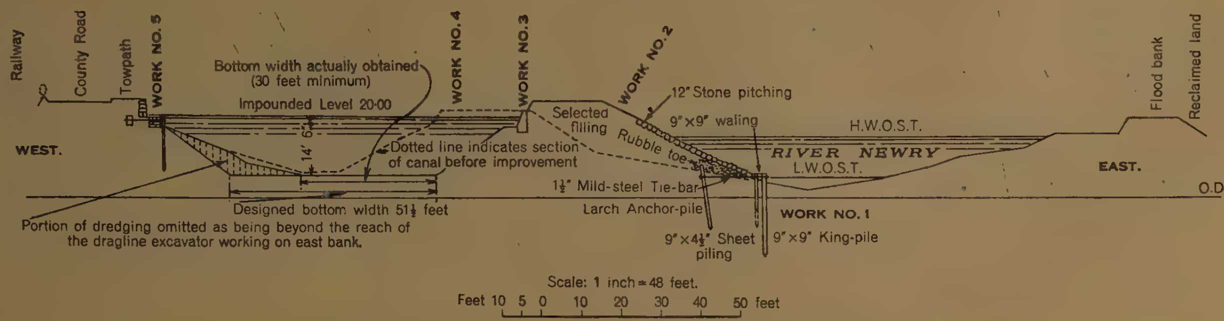
A third way was to obtain the necessary increased width of channel by excavation on the east, or river, embankment of the canal, and this method was the one that was adopted (*Fig. 3*). As the property of the Trust extends to the east bank of the tidal river no acquisition of land was necessary for the purpose of widening, but the design involved the partial reconstruction of the embankment between the ship-canal and the river Newry, and it was necessary to obtain the consent of the Board of Trade for the execution of the works that encroached on the tidal waters of the river.

Fig. 2.



CROSS SECTION OF RENNIE'S CANAL VICTORIA LOCK TO FATHOM.

Fig. 3.



CROSS SECTION OF OLD CANAL: FATHOM TO ALBERT BASIN.

The Improvement Scheme (1930).

The improvement scheme comprised the following works :—

(*a*) Widening, deepening, and general rectification of the fairway of the portion of the ship-canal between Fathom and Albert basin, together with ancillary works.
(*b*) Bank-protection works and the reconstruction of the overflow weir.
(*c*) New outer lock-gates at Victoria lock.
(*d*) New wharf and strengthening of the existing quay walls on the west side of Albert basin.

The installation of new lock-gates could not be carried out without the complete suspension of navigation, and a period of 28 days was allowed for the execution of this work. With this exception, all other works had to be executed in such a manner that the normal shipping-traffic of the port was uninterrupted, and this stipulation by the Harbour Trust not only influenced the design and method of construction of the various works, but also necessitated the adoption of a time-table so that all underwater-work in connection with each contract or section of the undertaking might be carried out during this period of 28 days.

For convenience of description, and to facilitate tendering, the scheme was divided into the following eight works :—

Work No. 1 The construction of a submerged breastwork of timber sheet-piling in the tidal waters of the river Newry, as a protection for Work No. 2.
Work No. 2 The partial reconstruction of the embankment between the ship-canal and the river Newry.
Work No. 3 Bank-protection works on the east or river bank of the ship-canal from Fathom to Albert basin, and the reconstruction of the overflow weir.
Work No. 4 The widening and deepening of the old portion of the ship-canal from Fathom to Albert basin.
Work No. 5 Bank-protection works on the west or road bank of the ship-canal from Fathom to Albert basin.
Work No. 6 The construction of a new wharf and the strengthening of the existing quay-walls on the west side of Albert basin.
Work No. 7 Repairs to both banks of the new portion of the ship-canal from Victoria lock to Fathom.
Work No. 8 The installation of new outer gates at Victoria lock.

The improvement scheme, which was estimated to cost £48,970

exclusive of engineering fees, was submitted to, and approved by, the Minister of Commerce, who indicated a Government Grant of half the actual cost of the scheme up to the limit of the approved estimate. The contract drawings and specifications were submitted to, and approved by, Mr. P. E. Shepherd, O.B.E., M. Inst. C.E. (Director of Works, N.I.) who is the Technical Adviser to the Ministry of Commerce.

Before embarking on the contract stage of the scheme an extensive examination of the bed and embankments of the old canal was carried out by means of trial pits and borings. No trace of rock was discovered in the bed of the canal, and there was no evidence that a puddle core had been used in the construction of the embankments.

Description of Works.

Bank-Protection Works.

The bank-protection works were the first to be put in hand, and as the widenings were being effected entirely on the east or outer bank, the west bank formed the base line from which the new line on the east bank was set out. For this reason the work on the west bank was carried out in advance of that on the east bank.

Bank-Protection Works on Road or West Side (*Work No. 5*).

The passage of vessels through the restricted portions of the canal at a speed of from 3 to 4 miles per hour causes severe wear on the banks. The wear is chiefly caused in two ways:—

(1) From a point about opposite the bow of the vessel the water-level in the canal commences to fall rapidly towards the mid-ship point, where it reaches its maximum drop of about from 6 to 9 inches, according to the size and speed of the vessel. No appreciable elevation of the water-level of the canal in front of the advancing vessel has been observed, so that this "draw-down" produces not only a certain amount of "squat" for which provision must be made in the depth of the waterway, but it also produces a very rapid "back-run" which sucks out the lighter material in the bank and consequently undermines the pitching.

(2) This "draw-down" is succeeded in the wake of the vessel by a "following wave" which rises about 9 inches higher than the normal water level in the canal, and breaks over the upper part of the pitching.

The canal lies on the line of the prevailing wind, which obtains a fetch of about 1 mile on several reaches of the canal, but the running wave formed causes little damage to the banks.

After careful investigation, it was concluded that in this canal the wave- or water-action referred to had little or no effect on the under-water slopes below a depth of about 3 feet from the surface. The pitching on both banks in the restricted portion of the waterway had been repaired from time to time by means of a facing of open timber piling, which was cut off at water-level and was surmounted by a stone-pitched slope. This form of construction, however, did not prove satisfactory, as the pumping action caused by the "draw-down" continued to remove the soft material behind the open sheet-piles and in a comparatively short time undermined the pitching, which again collapsed. The pitching was worked by masons and was exceedingly well set, but the cost of continual repair was excessive, and it was imperative that some simpler, cheaper and more effective method of bank-protection should be devised that would reduce maintenance-costs to the absolute minimum.

There is ample evidence that the waters of the canal have very little effect on timber that is completely immersed, and as there already existed long stretches of open timber piling in the form described along the west or road bank, it was considered economical to retain these structures if it were possible to adapt them to a new form of construction. It was also considered economical to use in a new form the large quantity of pitching-stones that already existed either loose or as intact pitching along the slopes of the bank. These two considerations resulted in the evolution of the design of bank-protection adopted for the west or road bank (*Fig. 4*).

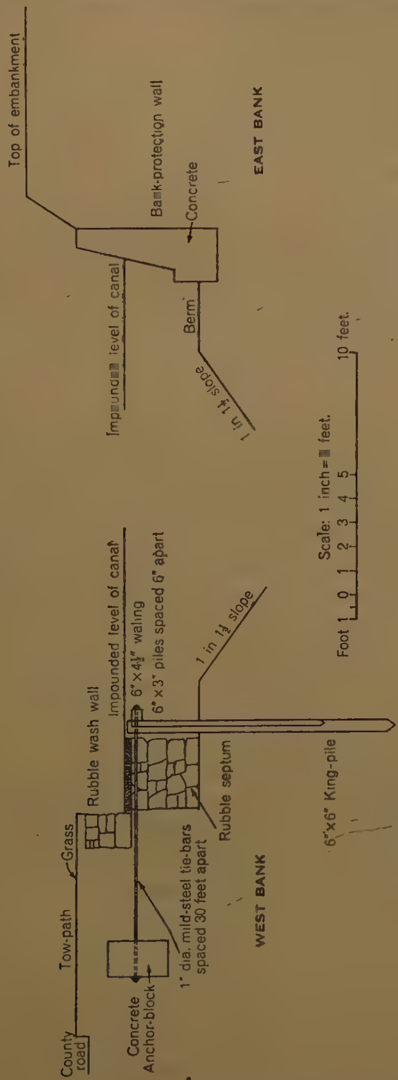
The intention of the design was to treat the open sheet-piling as a crib-work, and to interpose between the crib-work and the soft material in the face of the bank an independent septum, about 3 feet in width, of rubble filling, founded at a depth of about 3 feet below canal-level, where the water-action was negligible. It was a fundamental principle in the design that the septum should be flexible, so that it might accommodate itself, if necessary, to a yielding foundation. The work had to be carried out without lowering the working-level of the canal, and it was not practicable, therefore, to make a complete examination of the foundation upon which the septum rested.

The breadth of 3 feet enabled a considerable amount of cross-bonding of the stones to be effected, and provided a hard body of sufficient mass to resist the pumping action of the "draw-down." Moreover, as the septum is built independently of the crib-work, any settlement that may occur due to the foundation consisting of unfavourable material will take place in a vertical direction, and in that case the settlement can be made good by simply adding the requisite amount of rubble at the top instead of underpinning the

bottom. The crib-work merely safeguards the septum of rubble-filling from settling outwards, and acts as a second line of defence. All gaps between the existing piling were filled in a similar manner

Fig. 5.

Fig. 4.



WORK No. 3.

WORK No. 5.

with new piling so as to provide a continuous line of crib-work. The face of the bank above water-level is protected with a dry stone wall 18 inches thick and about 2 feet in height.

The new crib-work (*Fig. 4*, p. 61) was of native oak constructed as follows :—

Sheet-piles, 6 inches by 3 inches and from 8 feet to 10 feet long, spaced 6 inches apart in the clear.

King-piles, 6 inches by 6 inches and 10 feet long, spaced at 10 feet centres, every third pile being anchored back by a 1-inch-diameter mild-steel rod to an anchor-block sunk in the towpath.

Walings, 6 inches by $4\frac{1}{2}$ inches, in long lengths.

The total length of protection work executed on the west bank is 2,306 linear yards, and the work was carried out by direct labour.

Bank-Protection Works on East or River Side (*Work No. 3*).

The bank-protection works on the east bank (*Fig. 5*, p. 61) were carried out in an open trench behind that portion of the bank that was to be removed by dredging in the subsequent operation of enlarging and widening the waterway. As far as can be ascertained, the original outer bank was formed artificially, partly by cutting the lower portion of the waterway in the former natural foreshore, and partly by embanking the upper portion with imported material. The latter consists of light sandy loam which was apparently obtained from pits on the adjoining hillsides. The embankment, which was 20 feet in width at the top, stands about 7 feet above the average level of the former natural foreshore, which consists principally of fine alluvial clay or silt interspersed with shallow layers of sand. The natural clay-stratum of the old foreshore or marshland was found, therefore, about 5 feet below the present impounded level of the canal, and being beyond the influence of the wave and water-actions referred to, it formed a suitable foundation for the bank-protection structures. At the outset, it was intended to make use of the pitching-stones that already existed on the bank, and to protect the slopes with pitching, but after a length of about 500 feet had been completed it was found that, due to the confined space of the trench, the labour cost of pitching was excessive. With the exception of this trial length, the whole of the bank-protection on the east bank consists of a concrete wall in which the old pitching stones have been used as displacers.

For a distance of about 200 yards on the stretch between McShane's bridge and the boat-house the concrete wall was founded at a depth of 8 feet below the water-level of the canal, owing to the presence of sand in this part of the embankment. The subsequent widening operations necessitated the removal of this stretch of the embankment in its entirety, and it was reconstructed farther eastwards. A few portions of new work, where the new line was too close to the waterway to allow of the work being done in a trench,

were executed during the period of 28 days when navigation was suspended and the water-level in the canal was lowered by about 3 feet. The total length of bank-protection work executed on the east bank is 2,437 yards. The overflow weir was reconstructed in reinforced concrete with a timber gangway carried on angle-iron supports. This work was carried out by direct labour.

Bank-Repairs from Victoria Lock to Fathom (*Work No. 7*).

Certain portions of the outer bank (*Fig. 2*, p. 57) of the new canal, which had been constructed during 1842–1850, had settled appreciably, and as the water-level of the canal had been raised by about 1 foot the freeboard along these stretches had vanished. In fact at a few isolated spots the water was lipping over the top of the bank. The repairs, which were executed during the period of 28 days referred to, consisted mainly of making good slips in the pitching on both banks and in the construction of a concrete wash-wall along the low-lying portions of the outer bank. The work presented no special features and was carried out by direct labour.

Partial Reconstruction of the Embankment between the Canal and the river Newry, extending from McShane's Bridge to the Boat-House (*Works Nos. 1 and 2*).

The map (*Fig. 1*, p. 52) shows that the stretch of canal from McShane's bridge to the boat-house, a distance of approximately 700 yards, was the most critical section of the whole works. A narrow bank, 20 feet in width at the top, divided the canal from the river. The marsh-lands on the river side of the bank had been denuded and the base of the bank was exposed to the scouring action, not only of tides, but also of the river in flood. The trial pits and borings revealed the presence of layers of sand in the base of the bank and a fairly deep stretch of sand for a distance of about 200 yards north of McShane's bridge. The cross sections along that stretch showed that the waterway tapered from a surface width of 85 feet at the boat-house to a surface width of only 65 feet opposite McShane's bridge. The scheme provided for a minimum width at the surface of 92 feet, and, as has been stated already, the necessary widenings had to be effected entirely on the outer bank. A considerable portion of the embankment extending from McShane's bridge to the boat-house therefore had to be removed and a new embankment formed on the river side in order to provide the increased waterway.

Breastwork of Sheet-Piling (*Work No. 1*).

The first step in the reconstruction of the bank was the provision of a submerged breastwork of sheet-piling in the tidal waters of the

river Newry to form a protection for the toe of the new embankment.

The breastwork (*Fig. 3*, p. 57), which extends for a distance of 700 linear yards, consisted of a sheeting of 9-inch by 4½-inch native-oak piles closely driven in lengths of 8 and 12 feet according to their position and the nature of the strata. The longer piles were used at the northern end of the breastwork near the boat-house, where the set of the river induces the greatest erosion, and in such other places where the shorter piles proved the presence of pockets of soft material. The king-piles are of creosoted Oregon pine, 9 inches by 9 inches in section and 20 feet in length, and they are spaced 10 feet apart. The walings are also of creosoted Oregon pine, the outer walings being 9 inches by 9 inches in section and the inner waling 9 inches by 3 inches, bolted to the outer waling with ⅞-inch diameter bolts every 5 feet. The level of the underside of the waling is fixed at L.W.O.S.T. (about + 5 O.D.). The anchor-piles are of native larch, 12 inches diameter at butt and 16 feet in length, pitched raking, and spaced 30 feet apart (three bays). The tie-rods are of mild steel 1¼ inches in diameter, screwed up on 4½-inch by 4½-inch by ⅜-inch steel plates.

The king-piles were ordered by the contractor in 32-foot lengths and were driven in these lengths to the required depth, when the top 12 feet were cut off. The king-piles and anchor-piles were driven first from floating plant operating at high tide ; a barge carried the piling frame, winch and steam boiler. A drop-hammer weighing 2¼ tons was found more suitable than one of 30 cwts. for driving the king- and anchor-piles to the desired depths in the substrata of sand and glacial moraine.

The sheet-piles were driven at low water by a No. 5 McKiernan-Terry hammer worked from a travelling platform, which carried the winch and leaders and which rested partly on a rail fixed on the outer waling and partly on a rail laid on a berm cut on the outer slope of the existing embankment. The range of tide, which is about 11½ feet, and the large amount of mud deposited at each tide, retarded progress. The presence of boulders in the substrata prevented a close fit between the sheet piles, and tapered pieces or "slivers" were required at intervals. The work was carried out under contract by Messrs. John Graham, of Dromore, Co. Down.

Reconstruction of Embankment (*Work No. 2*).

The reconstruction of the embankment (*Fig. 3*, p. 57) followed closely upon the completion of each section of the breastwork, and was carried out mainly under tidal conditions. All mud and other soft material was removed from the bed of the tidal river forming the

base of the new work. A rubble toe was constructed to form a support for the earth filling and to act as a filter at low water. Selected dry spoil from the dredgings and excavations was used for the earth filling, which was spread in shallow layers, well rammed in the dry state, and each layer subjected to tidal action before the next layer was tipped. The existing stone pitching on the outer slope of the bank was removed as the work proceeded and the new filling was well bonded, in vertical steps, into the old earthwork. The action of successive tides on each layer had a very consolidating effect on the new filling, which never showed at any time any appreciable tendency to settle unequally or independently of the old earthwork. The new embankment as finished is 18 feet wide at the top, with a freeboard to the canal of 4 feet instead of 2 feet as formerly, the inner or canal slopes being 1½ to 1, and the outer or river slopes ranging between 1½ to 1 and 2 to 1. The river slopes are protected with pitching 12 inches deep, from the waling level (+ 5·00 O.D.) to 3½ feet above H.W.O.S.T. (+ 20·00 O.D.). This work was carried out by direct labour.

Widening and Deepening the Waterway of the Canal (Work No. 4).

As has been stated, the waterway of the canal was to be widened and deepened so as to provide a fairway with a depth of 14½ feet of water over a bottom width of 51½ feet. Having regard to the nature of the strata in which the channel was to be cut, it was concluded that the sides below a depth of 3 feet from the surface would be stable if trimmed to a slope of 1½ horizontal to 1 vertical. In order to provide a tolerance for the dredging of the side slopes and to meet the contingency of a flattening of the slopes where unfavourable substrata might be encountered, it was considered advisable to form a berm 3 feet in width between the top of the slope and the foundation of the concrete wall (*Fig. 5*, p. 61).

The maintenance of navigation created many difficulties in the execution of all the works, but especially so in the case of the dredging of the waterway. The conditions laid down by the Trust regulating the use of floating plant practically ruled out that method of carrying out the work, and after tenders had been obtained and full consideration had been given to various proposals, it was decided to purchase a dragline excavator and to carry out the work departmentally. The machine that was selected was a ½-cubic-yard type Priestman dragline excavator, petrol-paraffin driven, fitted with a 40-foot boom and equipped with ¼- and ½-cubic-yard buckets.

As the widenings were being made entirely on the east bank, the machine, working on that bank, was able to command the bulk of the

excavations, and, except on one stretch, could deposit the spoil on the marshlands immediately behind it without a second handling. On the stretch between McShane's bridge and the boat-house the spoil was deposited on the top of the embankment and removed, when dry, to the areas provided for spoil, or, where suitable, it was deposited as filling in the reconstruction of the embankment (work No. 2). Although the rate of progress was rather slow, the unit cost was found quite reasonable, having regard to the nature of the material which had to be removed and to the fact that the excavator was operating for long spells at the limit of its out-throw and digging depth. The excavator could not reach the underwater slopes on the west or road-bank side, and this dredging, which consists of the trimming of the slope and the smoothing out of the bottom width to the designed width of 51½ feet, has been deferred. In order to complete the dredging on the west side of the canal it would have been necessary to transfer the excavator to the tow-path on the west bank, and this step was not considered economical unless the second stage of the dredging work, that is, the subsequent dredging to 15½ feet to provide navigation for vessels of 800 tons gross, was also undertaken at the same time. On economical grounds, therefore, the postponement of the dredging on the west side was justified, and its omission (shown hatched in *Fig. 3*, p. 57) did not materially reduce the sectional area of the waterway.

Actually, a minimum depth of 14½ feet of water has been obtained over a minimum bottom width of 30 feet, whilst the remaining 21½ feet of the designed bottom width of 51½ feet enjoys the same navigation facilities that have prevailed hitherto. The fairway follows the line of the east bank and has been tested with very successful results for a minimum depth of 14½ feet of water over a minimum bed width of 30 feet by means of a " sweep " consisting of an angle-iron frame mounted on a timber raft. As the water-level in the canal does not vary more than a few inches the " sweep " constituted a severe test, which was positive and absolute.

A similar excavator working on the west or road bank should be able to effect the deepening to 15½ feet when the occasion arises, together with the rectification of the underwater slope on that bank and the smoothing out of the bottom width ; the spoil will, however, have to be transported by barge. The total amount of dredging carried out on the east bank amounted, by in situ measurement, to 70,000 cubic yards.

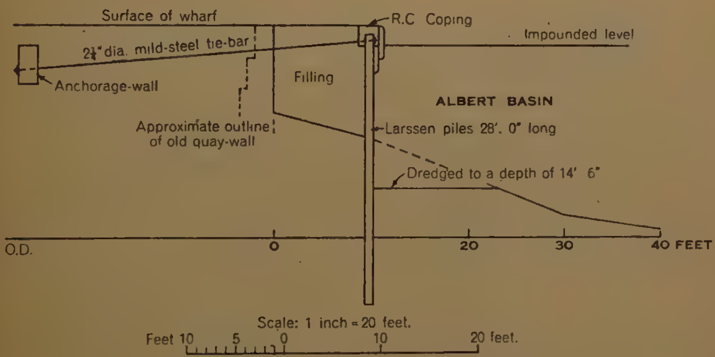
New Wharf, etc., Albert Basin (Work No. 6).

Three berths already existed along the coal quay on the west side of Albert basin, and additional berthing accommodation was

necessary. It was decided to construct a new wharf 600 feet long southwards of the existing berths so as to provide three additional berths, and to strengthen the quay-walls of the remainder of the water front northwards, including the existing berths, by means of stay-piles. The latter work was necessary because the walls were considered unsafe and the railway company would not permit their engines to travel on the outer line of the sidings for the purpose of shunting wagons or marshalling trains.

The new wharf (*Fig. 6*) was constructed with No. 3 Larssen steel piles, with a 0·3-per-cent. copper content; the piles were 28 feet in length, tied back every 10 feet with $2\frac{1}{4}$-inch-diameter mild-steel rods averaging 36 feet long into a continuous reinforced-concrete anchorage-wall, which was constructed in a trench in natural ground

Fig. 6.



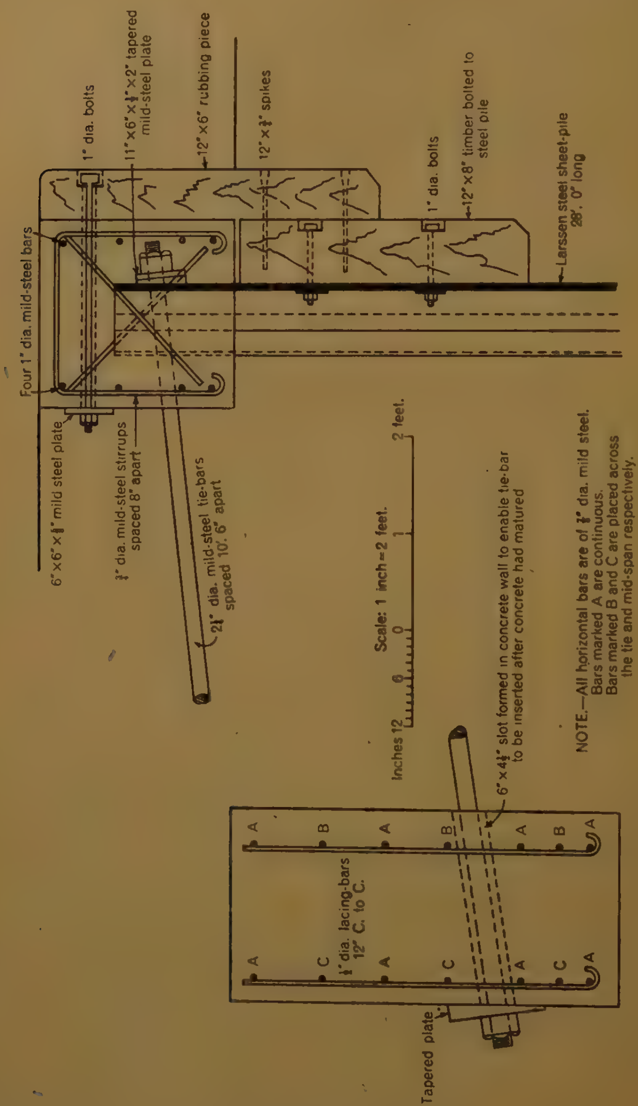
WORK NO. 6: NEW WHARF.

beyond the calculated angle of rupture of the soil. The anchorage-wall (*Fig. 7*, p. 68) is 2 feet in thickness and 4 feet in height, and is founded 6 feet below ground-level. The reinforcement consists of four continuous rows of $\frac{7}{8}$-inch-diameter mild-steel bars on each face and four additional rows of similar bars in 6-foot 6-inch lengths over the bearing-plates of the tie-rods and at the mid-spans. The vertical lacing-bars are of $\frac{1}{2}$-inch-diameter mild steel, and are spaced 12 inches apart. The bearing-plates of the tie-rods are 12 inches by 12 inches, tapered to suit the rake of the tie-rod, and averaging $1\frac{1}{2}$ inch in thickness.

The steel piling was surmounted by a reinforced-concrete coping, 2 feet square in section. The reinforcement consists of four 1-inch-diameter mild-steel bars on each face and $\frac{3}{4}$-inch-diameter mild-steel stirrups of triangular shape spaced at 8-inch centres (*Fig.* 7, p. 68). Each cast-iron bollard was carried on a cluster of four native-larch

piles, 26 feet in length, which supported a 4-foot 6-inch by 4-foot 6-inch by 1-foot 6-inch reinforced-concrete base. Two $1\frac{1}{2}$-inch-

Fig. 7.



Details of Anchorage-Wall and Coping.

diameter mild-steel tie-bars 16 feet in length placed diagonally take the pull on the bollard, which is independent of the steel piling. The space between the steel piling and the old quay-wall was filled

in with selected spoil, obtained partly from the general dredgings from the canal and partly from the dredgings from the new berths alongside the wharf.

The strengthening of the existing berths and the remainder of the quay walls for a distance of 513 linear feet northwards of the new wharf was carried out by means of 12-inch by 12-inch creosoted Oregon-pine stay-piles 22 and 24 feet in length, spaced 8 feet apart. Each stay-pile was tied back to a reinforced-concrete anchorage-wall similar to that described above, with a $1\frac{3}{4}$-inch-diameter mild-steel rod 32 feet long. Concrete in bags was placed between the toe of the existing wall and the back of the stay-pile, and a continuous waling with packing pieces was also fixed 3 feet below the impounded level. The work was carried out under contract by Messrs. John Graham, of Dromore, Co. Down.

New Outer Gates at Victoria Lock (*Work No. 8*).

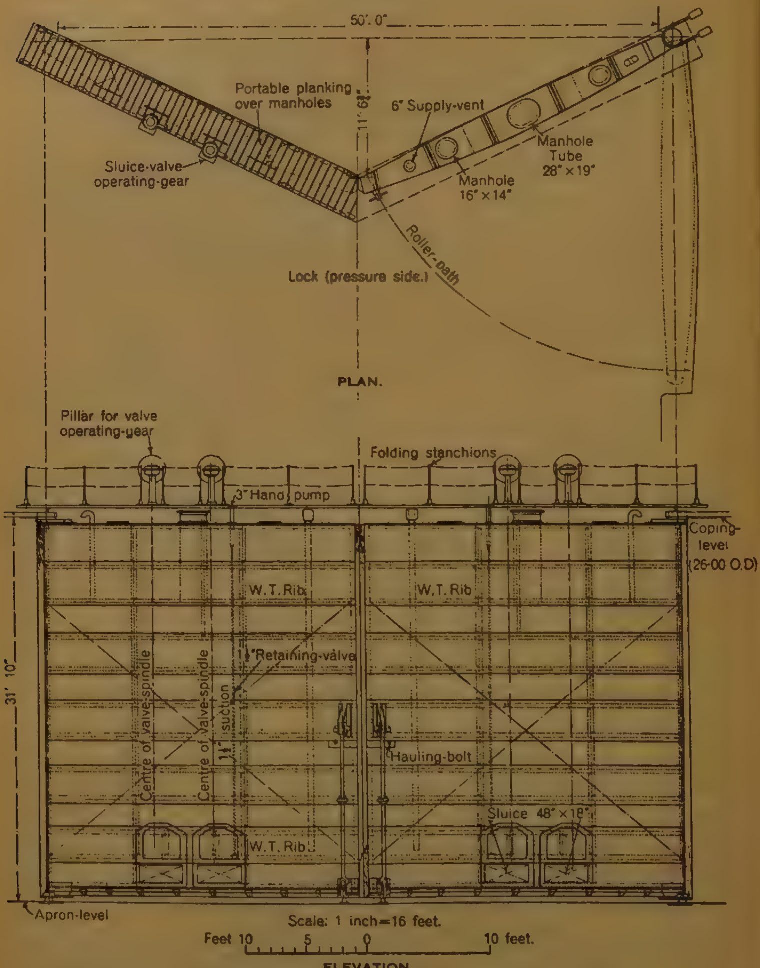
The timber outer gates at Victoria lock were over 80 years old, and were in need of a complete overhaul. The timber was generally in fair order considering its age and usage, but the iron fittings were badly corroded. Apart from the cost of the plant that would have been required for the purpose of effecting repairs, the length of time that such repairs would have taken would have been ruinous to the trade of the port; the only alternative was, therefore, the provision of new gates that could be quickly installed immediately the old gates were dismantled.

The new gates (*Figs. 8* and *9*) are constructed of mild steel with greenheart heel-posts, mitre-posts, and clapping-sills. The gates conform to the conditions determined by the details of the existing masonry structure. The length of each leaf is 29 feet and the rise is 11 feet $6\frac{3}{4}$ inches. The width of each leaf at the centre is 2 feet 9 inches and at the ends is 1 foot 4 inches. The back of the gate forms an arc of a radius of 92 feet 3 inches and the front of the gate forms an arc of a radius of 292 feet 8 inches. Bosom-pieces, which would have involved 5-ply riveting, were omitted, and instead the butts of the boundary-bars were electrically welded after the flange-angles had been riveted to the rib-plates.

The total weight of each leaf is approximately 48 tons and the buoyancy of each leaf (between its lowest and highest working levels) varies from 25 tons to 38 tons. No assurance could be given that the gates would not be operated during low water for purposes other than shipping, and it was decided to fit the gates with rollers suitable for use with the existing roller-paths. The roller-paths, which are $4\frac{1}{2}$ inches wide, were found, when examined by a diver,

to be worn unevenly, and the use of a conical type of roller would have involved the underwater dressing of the roller-paths. As the cast-iron nosing on the masonry sill had turned "spongy" in places

Figs. 8.

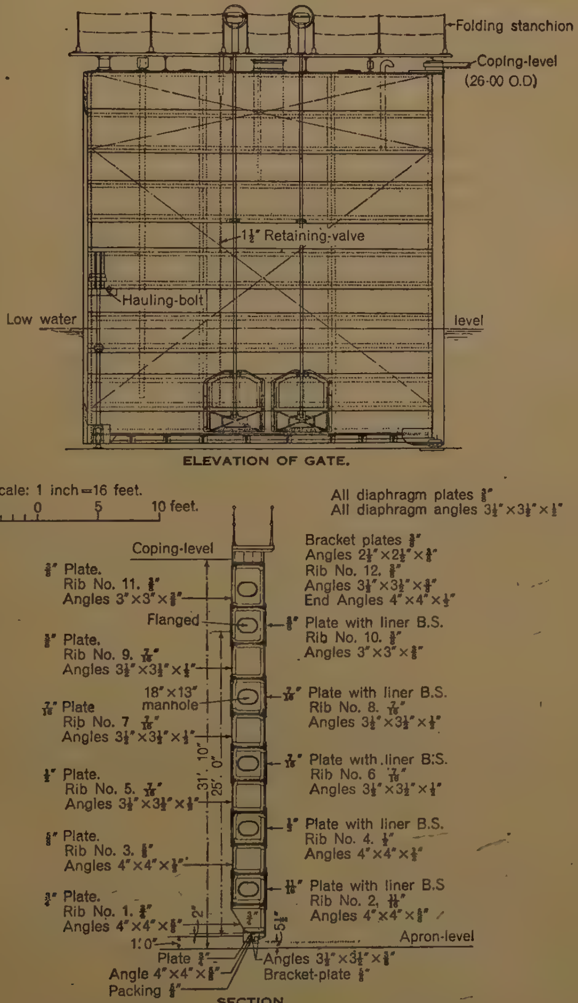


New Outer Gates at Victoria Lock.

it was concluded that the roller-path was not in a condition to stand chipping, and for this reason, together with the fact that the contingency of the gates being operated at low water was rather remote, it was decided to abandon the conical shape and instead to round off

the tread of the roller so as to provide a narrow width of bearing. The roller thus bears on the path at the two ends but rides about 1 inch clear in the middle of the path. The roller is 12 inches in

Figs. 9.



New Outer Gates at Victoria Lock.

diameter and 3 inches on tread, and is carried on a $2\frac{3}{4}$-inch axle-pin with $\frac{5}{8}$-inch bushings.

The gates are fitted with two sluices each 4 feet by 1 foot 6 inches in each leaf. The sluices are placed between ribs Nos. 1 and 2 and

are constructed with cast-iron frames and "Ferac" brand semi-steel doors, phosphor-bronze facing strips being fitted to frame and door. The four sluices enable the levelling operation at lowest working level to be completed in 10 minutes, including the time for opening. Each sluice is geared to be operated by one man who can open or close the door fully in $1\frac{1}{2}$ minute. The lock chamber is 220 feet long and 50 feet wide. The coefficient of discharge is about 0·6.

The gates were assembled on a site on the bank of the canal adjacent to Albert basin, and when the riveting had been completed they were tested under a hydrostatic pressure of 13 lbs. per square inch. Before launching the space between the sill girder and water-tight rib No. 2 was sheeted over and caulked in order temporarily to increase the buoyancy. The gates were then transferred from the erection platforms to launching ways, launched into the canal, the level of which had to be brought up to normal for this purpose, towed down the canal, then into the lock chamber, and finally stepped. The existing anchorages were used for the new gates, but the old collar-straps were very much worn where they were bearing against the old heel-posts, and as they were not capable of adjustment, new collar-straps were provided of a design that enabled minor adjustments of the gates to be made from time to time.

The actual time that the canal was out of commission for the installation of the lock gates was 34 days (including 5 Sundays) or 6 days in excess of the stipulated period. The contractors were Messrs. Vickers-Armstrong, Limited.

Additional Works.

Before the completion of the Canal Improvement Scheme the Harbour Trust authorized the carrying out of the undermentioned additional works.

Railway Siding at Coal Quay.

In order to facilitate the rapid discharge of cargoes, railway sidings were laid down on the new wharf with crossovers at each berthage, and the shunting neck was extended so as to accommodate 40 wagons. The railway work was carried out under contract by Messrs. John Graham.

Sluices at Victoria Lock.

A filling culvert was constructed about the year 1917 to replace the sluices in the old timber inner lock-gates, which were then in a bad state of repair. The old gates were replaced in the year 1924

by a pair of new steel lock-gates, but no sluices were provided in the new gates.

The sluices in the old gates enabled an efficient scour to be obtained throughout the entire length and breadth of the lock chamber, and prevented the accumulation of silt in it. The filling culvert discharges over a tumbling-bay through openings in the east-side wall of the lock chamber, and consequently the scour is not directed through the breadth of the chamber. A large deposit of mud quickly gathers on the eastern half of the chamber and prevents the locking of two boats side by side. The reinforced-concrete paddles controlling the culvert are counterbalanced and operated by chains, so that the paddles can be lifted in a very short time and levelling operations can be carried out in the space of a few minutes. On the other hand, this type of gear depends on the preponderance of the weight of the paddle for shutting down, and this preponderance is not sufficient to overcome the frictional resistance created by the water-pressure until the levelling operations are completed. Thus no positive control was available in case of emergency when ships were being locked, and for the same reason it was not possible to use the sluices for flushing the lock chamber. Moreover, the paddles were not watertight, and the leakage from this cause, together with the quantity of water contained in the culvert, which was lost at each locking, was a serious drain on the inland water-reserves in times of prolonged drought.

The filling culvert was completed in 1918 and in 1922 the mud-problem became so acute that the Harbour Trust had to dredge the lock chamber at a cost of £600. In 1932 the mud had to be removed again at a cost of £400 and in another year the mud had accumulated to the same extent, which was about 2,000 tons. Furthermore, the approach channel in the estuary from Victoria basin to Victoria lock was becoming silted due to the absence of sluicing facilities at Victoria lock.

It was decided to fit penstocks on each of the four openings in the side wall of the chamber, and to install a system of sluicing jets housed in one of the recesses that had been provided originally for a pair of intermediate lock-gates. The sluicing jets are fed from the canal by a 24-inch-diameter concrete pipe which is controlled at its intake by a 24-inch-diameter penstock. The jets, which are 6 inches in diameter, are placed at the level of the gate-platforms and are operated at low tide under a difference in head of about 20 feet.

The penstocks were manufactured and erected under contract by Messrs. J. Blakeborough & Sons, Ltd., of Brighouse, Yorks. The structural work in connection with the installation of the penstocks

and the construction of the sluicing system was executed by direct labour. The total cost of the installation compares very favourably with the cost of an interim dredging operation, and, in view of the improvement that has already been effected, it is confidently hoped that the accumulation of mud in the lock chamber and approach channel will be prevented by a systematic use of the sluicing arrangements.

Conclusion.

Work on the scheme was commenced early in July, 1931, and the scheme was completed early in April, 1935. The cost of labour and materials remained practically unchanged throughout this period. As most of the work was carried out under rather abnormal conditions some details of cost may be of interest, and these are contained in the Appendixes.

Mr. H. P. McKenna, B.Sc., Assoc. M. Inst. C.E., acted in the capacity of Resident Chief Engineer from the commencement to the completion of the works, and to him is due the greatest credit for his successful organization and direction of the works carried out by direct labour.

The Paper is accompanied by four sheets of drawings, from which the Figures in the text have been prepared, by twenty-four photographs, and by the following Appendixes.

APPENDIX I.

Constructional Costs.

A. Works carried out under Contract.

Work No. 1. 715 linear yards of submerged breastwork in the tidal portion of the river Newry.

	£
Insurances, preliminaries, etc.	488
Excavation	344
King-piles	1,100
Outer waling	360
Sheet-piling	3,430
Inner waling	202
Larch anchor-piles	290
Tie-rods	85
Total	£6,299

Work No. 6. New wharf and strengthening of quay wall, etc., in Albert basin.

	£
613 linear feet of new wharf, including anchorage-wall, tie-rods, coping, fenders, filling and surfacing	6,659
Ten bollards complete on piled foundations	450
510 linear feet of strengthening quay wall, including stay-piles, anchorage-wall, waling, fenders and tie-rods	2,126
Sundry walings, concrete in bays, repairs, chocks, etc., carried out when water-level was lowered	330
Temporary rail-sidings, etc.	186
Total	£9,751

Work No. 8. Installing one pair of new steel outer lock-gates at Victoria lock. Entrance 50 feet wide and 31 feet high.

One pair of cellular gates constructed of mild steel, each leaf being 28 feet 3 inches long by 30 feet 3 inches high, including four sluices complete with gearing, gangway, roller and spindle together with greenheart sills, mitre- and heel-posts, but excluding roller-path, anchorages, collar-straps, and operating gear	£7,078

B. Works carried out by Direct Labour.

Work No. 2. Reconstruction of 715 linear yards of embankment.

Summary of Work.

3,000 cubic yards of excavation.
14,000 „ placing selected filling.
5,000 „ rubble toe.
7,000 „ pitching.

Summary of Cost.

	£
Wages	2,781
Material	766
Insurances	127 (Apportioned)
Equipment, etc.	101 (Apportioned)
Total	£3,775

Work No. 3. Bank-protection works on east bank.

Summary of Work.

2,437 linear yards of concrete wall.
570 „ repairs to existing pitching.
100 „ reconstruction of overflow-weir.

Summary of Cost.

	£
Wages	3,163
Material	2,459
Insurances	119
Equipment, etc.	157
Total	£5,898

Work No. 4. Widening and deepening canal by dredging.

Summary of Work.

70,000 cubic yards by drag-line excavator.
1,647 „ by grab-dredger (floating).

Summary of Cost.

	£	
Wages	2,524	
Material	1,383	
Insurances	114	(Apportioned)
Equipment	143	(Apportioned)
Plant	1,223	(Net)
Total	£5,387	

Work No. 5. Bank-protection works on west bank.

Summary of Work.

2,306 linear yards of piled cribwork and rubble walling.
792 „ repairs to existing pitching.

Summary of Cost.

	£	
Wages	2,832	
Material	1,520	
Insurances	100	(Apportioned)
Equipment, etc.	122	(Apportioned)
Total	£4,574	

Work No. 7. Repairs to pitching, etc., Victoria lock to Fathom.

Summary of Work.

1,403 linear yards on east bank.
904 „ west bank.
464 „ wash-wall or coping on east bank.

Summary of Cost.

	£
Wages	378
Material	283
Insurances	18
Equipment	20
Total	£699

Work No. 8. Additional work in connection with installation of new outer gates at Victoria lock.

	£
New collar-straps	48
Dredging lock chamber	366
Chipping and painting internal surfaces of outer and inner steel lock-gates	70
Total	£484

C. *Additional Works.*

Railway Siding on New Wharf, etc., carried out with second-hand relayable material at a cost of £1,195.

Summary of Work.

160	linear yards of	shunting neck.
400	"	sidings, including five crossings.
250	"	sluing existing track and three crossings.
1,250	"	surfacing track.

New Sluices on Filling Culvert at Victoria Lock.

	£
Installation of four sluices 3 feet 6 inches by 3 feet 3 inches under contract	359
Structural alterations and preparatory work carried out by direct labour	303
Sluicing system carried out by direct labour	140
Total	£802

APPENDIX II.

Analysis of Costs.

Note.—Wages 10*d.* per hour. Native oak 2*s.* 4*d.* per cubic foot. Oregon pine 2*s.* 8*d.* per cubic foot. Rubble 3*s.* 9*d.* per cubic yard. Broken stones 5*s.* 6*d.* per cubic yard.

The direct-labour costs set out below include foremanship, but do not include supervision and overhead charges.

Work No. 1. (Contract.)	King-piles driven complete . .	9*s.* per cubic foot.
	Sheet-piles complete	3*s.* per square foot.
	9-inch by 9-inch walings fixed complete	3*s.* per linear foot.
Work No. 3. (Direct labour.)	Excavation, including pumping, etc.	6*s.* per cubic yard.
	Concrete	28*s.* per cubic yard.
	Bank-protection wall complete .	43*s.* per linear yard.

Work No. 4. (Direct labour.)	Dredging by grab-dredger mounted on barge and removing spoil ashore (including hire of plant)	4*s.* 6*d.* per cubic yard.
	Dredging by drag-line excavator (including wages, fuel, repairs, and depreciation on plant) . .	1*s.* 4*d.* per cubic yard.

Note.—Material dredged below designed bed-level is not measured.

Work No. 5. (Direct labour.)	Native-oak piles dressed and driven by hand	6*s.* 4*d.* per cubic foot.
	6-inch by 4½-inch walings fixed .	1*s.* per linear foot.
	Excavation (partly under water) .	7*s.* per cubic yard.
	Rubble walling (old pitching stones re-used)	6*s.* 6*d.* per cubic yard.
	Bank-protection work complete .	38*s.* 6*d.* per linear yard.
Work No. 6. (Contract.)	Larssen steel piling (driven) . .	£17 per ton, or 5*s.* per square foot.
	Reinforced-concrete anchorage-wall	40*s.* per linear foot.
	Reinforced-concrete coping . .	15*s.* per linear foot.
Work No. 8. (Contract.)	Steel lock-gates, fabricated, transported, assembled, tested, launched and stepped (including sluices)	£4·1 per square foot.

Paper No. 5062.

"Simple Experimental Solutions of Certain Structural Design Problems."

By Professor ALFRED JOHN SUTTON PIPPARD, M.B.E., D.Sc., M. Inst. C.E., and STANLEY ROBERT SPARKES, M.Sc.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

INTRODUCTION.

IN 1925 Professor G. E. Beggs described an experimental method for determining the resultant action at different points in a loaded structure,[2] and since that date a number of other methods have been developed with the object of analysing stresses by mechanical means. These methods fall into two main categories, in one of which the object is to determine the influence-lines,[3] while in the other a direct measurement of slopes and deflections in a model are made.[4] In order to comply strictly with the mathematical requirements of the problem the displacements of the model structure must be kept very small, and this necessitates the use of accurate measuring instruments such as micrometer microscopes. This is a disadvantage if such mechanical methods are to be used for routine work in the design office, and it appeared to the Authors that, although mathematically the displacements should be made very small, it ought

[1] Correspondence on this Paper can be accepted up to the 15th March, 1937, and will be published in The Institution Journal for October, 1937.—SEC. INST. C.E.

[2] Prof. G. E. Beggs, "An Accurate Mechanical Solution of Statically Indeterminate Structures by Use of Paper Models and Special Gauges." Proc. Am. Concrete Inst., vol. xviii (1922).

[3] Otto Gottschalk, "Mechanical Calculation of Elastic Systems." Journal Franklin Inst., vol. 202 (1926), p. 61.

[4] Prof. J. F. Baker, "The Mechanical and Mathematical Stress Analysis of Steel Building Frames." Inst. C.E. Selected Engineering Paper No. 131.

to be possible to obtain good results even if they were made of such magnitude as would enable them to be measured without special instruments.

The object of the present Paper is to demonstrate that this assumption is a reasonable one and that influence-lines can be obtained with a good degree of accuracy without the use of special apparatus. This fact makes it possible either to check elaborate calculations very quickly by means of a model cut from xylonite, or to obtain approximate values of redundant reactions for preliminary design purposes.

The Authors wish to emphasize that there is nothing new in principle in the method described, which is that used by Professor Beggs, but whereas he distorted his models by very small amounts, using for this purpose an instrument which he called a "deformeter,"

Fig. 1.

they dispense with all such apparatus and obtain results of sufficient accuracy for many design purposes. One advantage of the present method is that errors due to temperature-effects and creep are negligible, whereas they introduce serious experimental difficulties when the displacements imposed upon the model are of microscopic size.[1]

Theory of Method Employed.

The method is based upon a well-known theorem enunciated by Professor Müller-Breslau,[2] which may be demonstrated here for completeness.

Suppose that A, B, and C are supports of a continuous structure and that it is required to obtain the influence-line of reaction for the support C. This support is supposed to be removed and to be replaced by a unit load acting vertically. Let the deflected form of the bottom chord ABC, which is known as the deflection polygon for that chord, be as shown in *Fig. 1*. It may be obtained either by direct calculation, by drawing a Williot—Mohr[3] deflection diagram,

[1] C. H. Lobban, "Mechanical Methods of Solution of Stresses in Frames." Trans. Inst. Engineers and Shipbuilders in Scotland, vol. lxxvii (1933–34), p. 169.

[2] "Festigkeitslehre und der Statik der Baukonstruktionen," p. 42. Leipzig, 1886.

[3] "Notions Pratiques sur la Statique Graphique." *Ann. Gen. Civ.*, vol. vi (Second Series, 1877), pp. 601 and 713.

or, as will be shown later, by experimental means. Since the diagram represents the deflection of the chord at all points it is also the influence-line of deflection for the point C, so that if a load of unity be placed at any point M on the bottom chord the deflection produced at C will be the ordinate, δ, of the deflection polygon at M.

If a unit force at C produces a deflection Δ at that point, it is necessary, in order to restore C to its original position, to apply a reactive force of $\frac{\delta}{\Delta}$.

Hence, if the ordinates of the deflection polygon be divided by Δ the resulting figure is the influence-line for the reaction at C. This is the general principle involved and is capable of many applications.

Experimental Procedure.

In the method to be illustrated a model of the structure is cut out of sheet xylonite and fastened down on a sheet of smooth paper in a manner appropriate to the support-conditions of the actual structure. The positions of a number of points on the model are marked by a sharp pencil, or in some instances by a fine point, on the paper, and a known displacement is then given to the point at which the redundant reaction is applied. This displacement is of such magnitude that it can be measured by an ordinary finely-divided scale.[1] The new positions of the original points taken on the model are marked and the displacements of these in appropriate directions are scaled. These displacements, divided by the displacement given to the redundant reaction, provide points on the influence-line. The procedure is best explained by a series of examples which will enable the method to be used in a variety of problems common in structural design.

Examples.

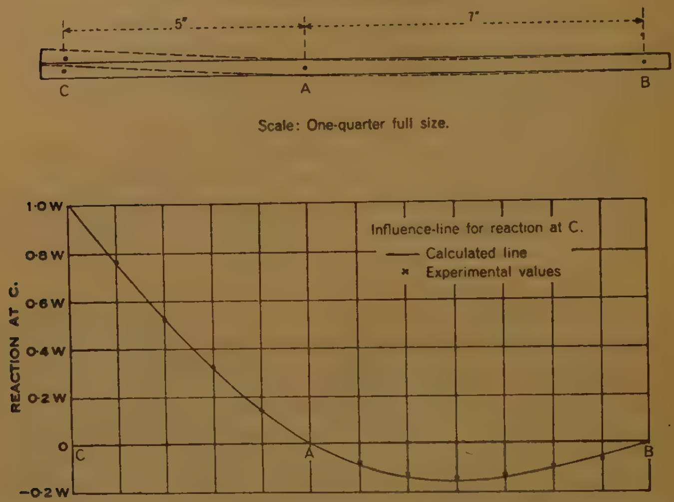
1. *Influence-line for reaction in a continuous beam.*

The first case is that of a continuous beam, which is given as the simplest possible illustration. A small strip of xylonite of uniform cross-section (as shown in *Figs. 2*, p. 82), was pinned at the three supporting points C, A, and B to a sheet of paper on a drawing-board, and a sharp pencil used to mark the position of the upper edge of the beam. The pin at C was removed, the point displaced by a small amount, and the pin replaced. The new position of the beam was then marked and the beam removed from the paper. The vertical distance between the two curves at any point when divided by the

[1] In all the examples given the unit of displacement was $\frac{1}{48}$ inch.

vertical displacement given to C gave a point on the influence-line for the reaction at C, and a sufficient number of such points were calculated and the required influence-line was plotted. The experi-

Figs. 2.



mental points are shown in *Figs. 2* together with the calculated influence-line, and the agreement is seen to be very good. The experimental and calculated data are given in Table I.

TABLE I.—CONTINUOUS BEAM.

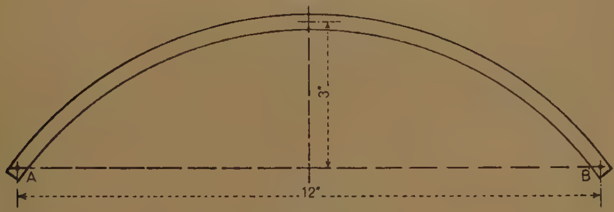
Distance from *C*: inches.		0	1	2	3	4	5	6
Vertical displacement.		10·7	8·1	5·6	3·4	1·5	0	− 0·9
Re-action.	Experimental.	1	0·76	0·52	0·32	0·14	0	− 0·08
	Calculated.	1	0·76	0·53	0·32	0·14	0	− 0·093

Distance from *C*: inches.		7	8	9	10	11	12
Vertical displacement.		− 1·4	−1·6	−1·4	−1·1	0·8	0
Re-action.	Experimental.	−0·13	−0·15	−0·13	−0·10	−0·07	0
	Calculated.	− 0·143	−0·157	−0·143	−0·107	−0·057	0

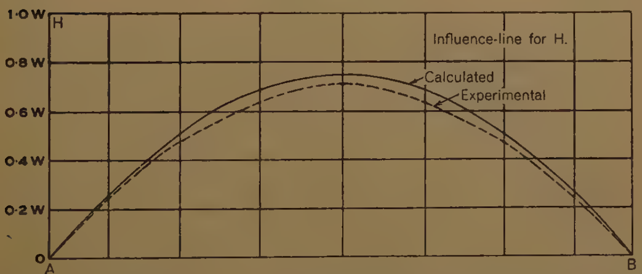
2. *The Influence-Line of Thrust for a Two-Pinned Segmental Arch.*

An arch of uniform cross section with a ratio of rise to span of 1 to 4, having the dimensions shown in *Figs. 3,* was pinned at points A and B to the sheet of paper. In this case the most satisfactory method of marking was to make fine scratches on one side of the model normal to the paper, and to use them to guide a pin point to the paper.

Figs. 3.



Scale: One-quarter full size.



The initial positions of the scratches on the model were marked, and the pin at B was then removed and given a small displacement in the direction BA. The new positions of all points around the model were then marked. On removing the model from the paper the vertical distances between the marks at each point were measured,

TABLE II.—SEGMENTAL ARCH.

Angle subtended by arch at centre = ϕ. Horizontal displacement of $B = 12{\cdot}5$.

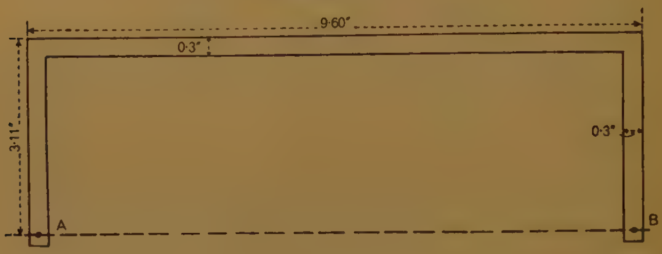
	Angular displacement from A.	0	$\frac{\phi}{8}$	$\frac{\phi}{4}$	$\frac{3\phi}{8}$	$\frac{\phi}{2}$	$\frac{5\phi}{8}$	$\frac{3\phi}{4}$	$\frac{7\phi}{8}$	ϕ
	Vertical displacement.	0	3·0	5·8	7·9	8·8	7·8	5·7	2·9	0
H	Experimental.	0	0·24	0·47	0·63	0·71	0·62	0·46	0·23	0
	Calculated.	0	0·261	0·508	0·688	0·746	0·688	0·508	0·261	0

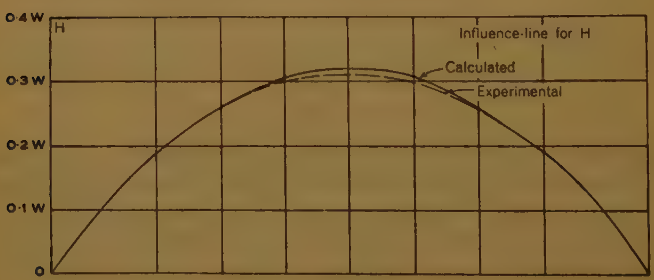
and divided by the displacement given to the point B. These values, given in Table II, are points on the experimental influence-line for the thrust of the arch, and are compared with the theoretical line in *Figs.* 3.

3. *Portal, Variable-Section Arch-Rib, and Spandrel-Braced Arch.*

The procedure and method of marking the model of the segmental arch was used in similar experiments to determine the influence-line

Figs. 4.



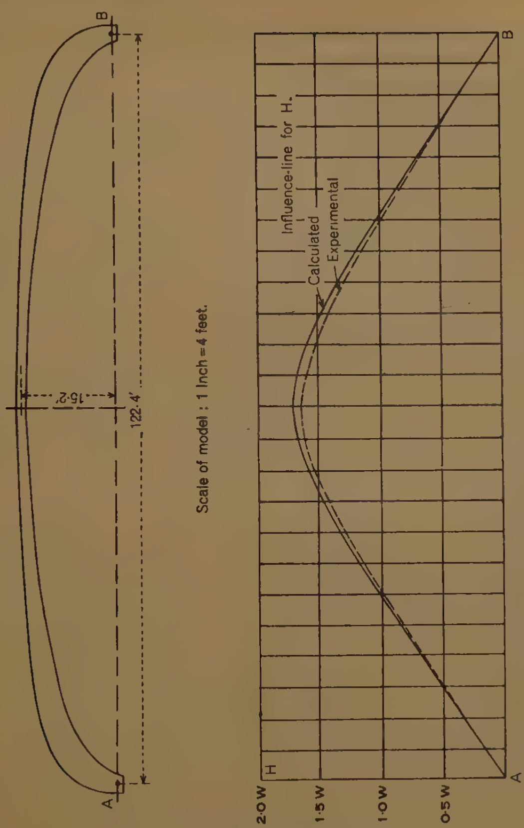


PORTAL.

for *H* for a two-pinned portal, a two-pinned arch of variable moment of inertia, and for a two-pinned spandrel-braced arch. Details of these structures are shown in *Figs.* 4, 5, and 6, together with the theoretical and experimental influence-lines for *H*. The data from which the curves were plotted are contained in Tables III, IV, and V.

In the case of the arch having variable moment of inertia, the depth of the model arch-rib was made proportional at every section to the cube root of the actual moment of inertia in the real bridge. The spandrel-braced arch is the Lengue bridge described by Mr.

Figs. 5.



VARIABLE-SECTION ARCH-RIB.

TABLE III.—PORTAL.

Horizontal displacement of $H = 10{\cdot}6$.

Distance from left-hand support : inches.		0	1·65	2·65	3·65	4·65	5·65	6·65	7·65	9·3
Vertical displacement.		0	2·0	2·7	3·2	3·3	3·2	2·6	2·0	0
H	Experimental.	0	0·19	0·26	0·30	0·31	0·30	0·25	0·19	0
	Calculated.	0	0·19	0·26	0·31	0·32	0·31	0·26	0·19	0

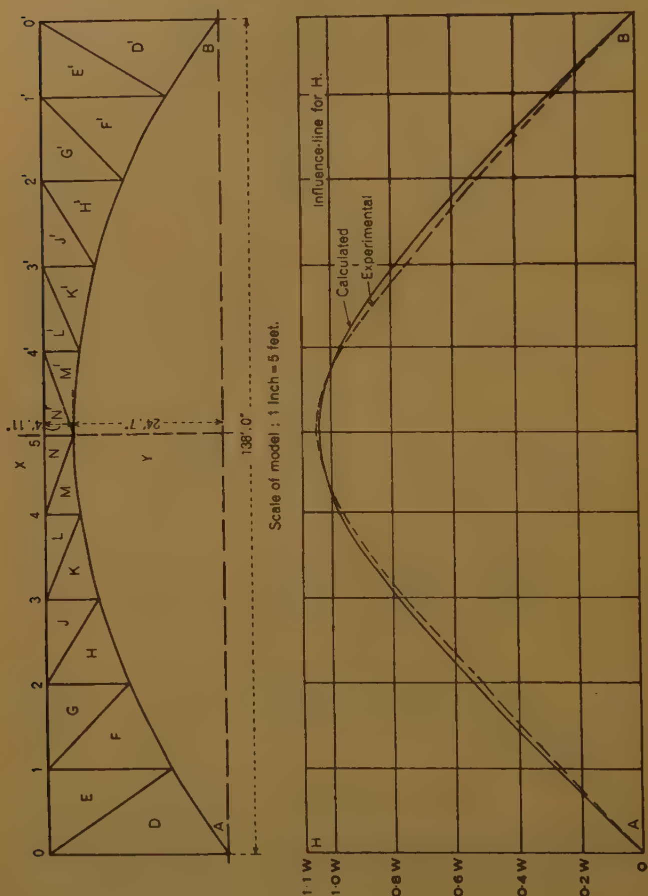
Ralph Freeman [1], and the experimental influence-line is compared with the calculated one given by him. The forces in the members of

[1] "The Design of a Two-hinged Spandrel-braced Steel Arch." Minutes of Proceedings Inst. C.E., vol. clxvii (1906–7, Part I), p. 343.

this structure are considered to be entirely axial, and so the areas of the bars of the model were made proportional to the areas of the actual structural members.

Having obtained such satisfactory results for pinned structures,

Figs. 6.



LENGUE ARCH.

the Authors were encouraged to a study by similar simple methods of the end fixing moments in encastré arches.

A second model of the segmental arch was made, at the springings of which were extensions stiffened by No. 18 S.W.G. duralumin plate to transmit bending about the pins A and B (*Figs.* 7, p. 89). Small

holes were drilled at C and D, the encastré condition of the arch being obtained when the model was pinned to the board at A, B, C, and D. By moving the pin C to C′ an angular rotation θ is imposed about the pin A. It is easily shown from the Müller-Breslau theorem that, if δ is the vertical displacement of any point on the arch due to such rotation at A, the ordinate of the influence-line of fixing moment

TABLE IV.—VARIABLE-SECTION ARCH-RIB.

Scale 1 inch = 4 feet. Horizontal displacement of H = 9·7.

Distance from A: feet.	Height to neutral axis: feet.	Moment of inertia, I: feet^4 units.	$\sqrt[3]{I}$.	Width of model: inches.	Vertical displacement.	H	
						Experimental.	Calculated.
0	0	—	—	—	0	0	0
5·1	7·75	27·05	3·000	1·50	1·6	0·16 W	0·16 W
10·2	10·25	13·30	2·370	1·185	3·1	0·32 W	0·33 W
15·3	11·33	8·62	2·050	1·025	4·8	0·50 W	0·51 W
20·4	12·15	5·75	1·792	0·895	6·3	0·65 W	0·68 W
25·5	12·80	3·78	1·558	0·779	7·9	0·82 W	0·86 W
30·6	13·42	2·45	1·348	0·675	9·7	1·00 W	1·02 W
35·7	13·92	1·78	1·212	0·605	11·0	1·14 W	1·19 W
40·8	14·25	1·22	1·070	0·535	12·5	1·29 W	1·34 W
45·9	14·50	0·82	0·936	0·468	13·6	1·41 W	1·48 W
51·0	14·80	0·53	0·810	0·405	14·7	1·52 W	1·59 W
56·1	15·00	0·38	0·725	0·362	15·5	1·60 W	1·67 W
61·2	15·20	0·278	0·653	0·327	16·0	1·65 W	1·70 W
66·3	15·00	0·38	0·725	0·362	15·5	1·60 W	1·67 W
71·4	14·80	0·53	0·810	0·405	14·6	1·51 W	1·59 W
76·5	14·50	0·82	0·936	0·468	13·6	1·41 W	1·48 W
81·6	14·25	1·22	1·070	0·535	12·5	1·29 W	1·34 W
86·7	13·92	1·78	1·212	0·605	10·9	1·13 W	1·19 W
91·8	13·42	2·45	1·348	0·675	9·6	0·99 W	1·02 W
96·9	12·80	3·78	1·558	0·779	7·9	0·82 W	0·86 W
102·0	12·15	5·75	1·792	0·895	6·3	0·65 W	0·68 W
107·1	11·33	8·62	2·050	1·025	4·9	0·51 W	0·51 W
112·2	10·25	13·30	2·370	1·185	3·4	0·35 W	0·33 W
117·3	7·75	27·05	3·000	1·50	1·6	0·16 W	0·16 W
122·4	0	—	—	—	0	0	0

at that point is $\frac{N\delta}{\theta}$, where N is the scale of the model (that is, 1 inch = N feet).

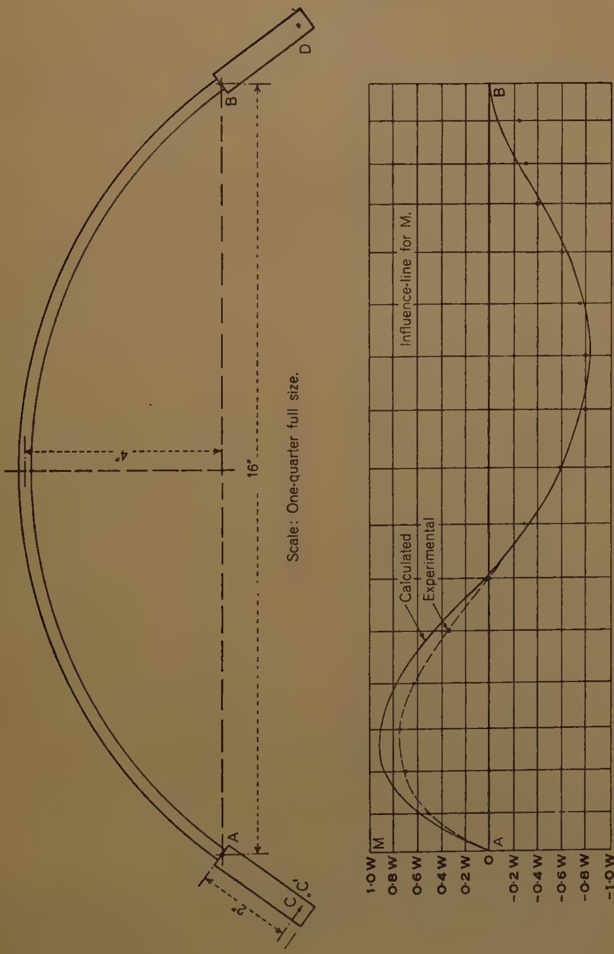
Experimental values thus found, and given in Table VI, were smaller than those obtained theoretically, but the position of the load to produce no bending moment at the abutment was determined very accurately. The error appears to be due to friction between the model and the paper due to slight buckling of the model. Experimental and theoretical influence-lines for this case are shown in *Figs.* 7, p. 89.

TABLE V.—LENGUE ARCH.

Scale 1 inch = 5 feet. Horizontal displacement of H = 10·0.

Member.	Cross sectional area.	Width of model-members.	Joint.	Vertical displace-ment.	H Experi-mental.	H Calculated.
EX	15·3	0·153	0	0	0	0
GX	15·3	0·153	1	2·6	0·26	0·28
JX	15·3	0·153	2	5·1	0·51	0·54
LX	21·2	0·212	3	7·6	0·76	0·78
NX	21·2	0·212	4	9·6	0·96	0·97
N′X	21·2	0·212	5	10·5	1·05	1·04
L′X	21·2	0·212	4	9·6	0·96	0·97
J′X	15·3	0·153	3	7·4	0·74	0·78
G′X	15·3	0·153	2	5·2	0·52	0·54
E′X	15·3	0·153	1	2·6	0·26	0·28
DY	30·4	0·304	0	0	0	0
FY	22·9	0·229				
HY	22·9	0·229				
KY	22·9	0·229				
MY	21·2	0·212				
M′Y	21·2	0·212				
K′Y	22·9	0·229				
H′Y	22·9	0·229				
F′Y	22·9	0·229				
D′Y	30·4	0·304				
XD	15·3	0·153				
EF	15·2	0·152				
GH	14·5	0·145				
JK	13·7	0·137				
LM	12·9	0·129				
NN	—	—				
M′L′	12·9	0·129				
J′K′	13·7	0·137				
H′G′	14·5	0·145				
F′E′	15·2	0·152				
XD′	15·3	0·153				
DE	10·3	0·103				
FG	10·3	0·103				
HJ	10·3	0·103				
KL	11·4	0·114				
MN	21·2	0·212				
N′M′	21·2	0·212				
L′K′	11·4	0·114				
J′H′	10·3	0·103				
G′F′	10·3	0·103				
E′D′	10·3	0·103				

Figs. 7.



ENCASTRÉ SEGMENTAL ARCH.

TABLE VI.—ENCASTRÉ SEGMENTAL ARCH.

Radius 10 inches. Angle subtended by arch at centre = ϕ.

Angular displacement from *A*.		Vertical displacement.	*M*	
			Experimental.	Calculated.
0		0	0	0
	$\frac{\phi}{16}$	1·8	0·52	0·58
$\frac{\phi}{8}$		2·2	0·69	0·87
	$\frac{3\phi}{16}$	2·6	0·75	0·90
$\frac{\phi}{4}$		2·2	0·63	0·79
	$\frac{5\phi}{16}$	1·2	0·34	0·47
$\frac{3\phi}{8}$		0	0	0·05
	$\frac{7\phi}{16}$	− 1·0	− 0·29	− 0·32
$\frac{\phi}{2}$		− 2·1	− 0·60	− 0·59
	$\frac{9\phi}{16}$	− 2·8	− 0·80	− 0·77
$\frac{5\phi}{8}$		− 2·8	− 0·80	− 0·84
	$\frac{11\phi}{16}$	− 2·6	− 0·75	− 0·79
$\frac{3\phi}{4}$		− 2·1	− 0·60	− 0·65
	$\frac{13\phi}{16}$	− 1·4	− 0·40	− 0·44
$\frac{7\phi}{8}$		− 1·0	− 0·29	− 0·23
	$\frac{15\phi}{16}$	− 0·9	− 0·26	− 0·07
ϕ		0	0	0

CONCLUSIONS.

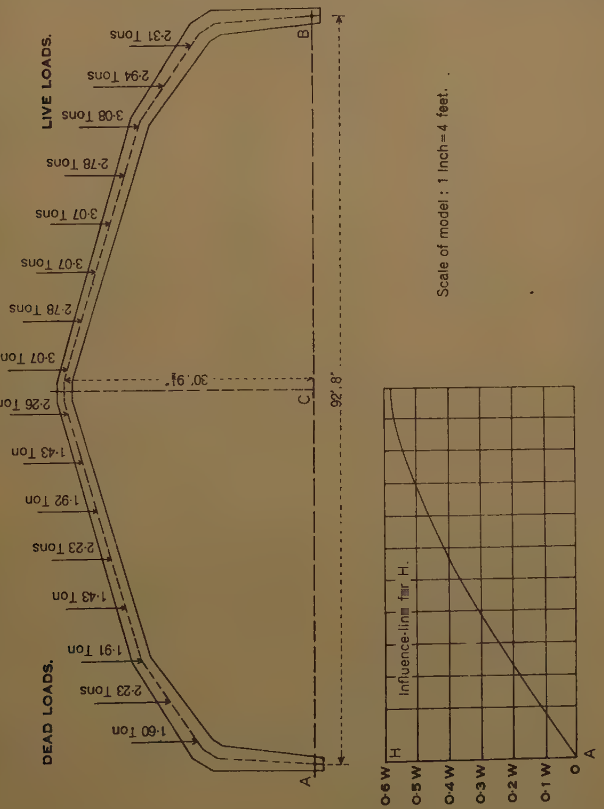
By the methods described, experimental influence-lines can easily be obtained correct to within a few per cent. of the theoretical values for such structures as beams, two-pinned arches, and portals, and the models for this purpose are easily and quickly made. An ordinary fretsaw was used for rough shaping and a file for finishing the models to within about $\frac{3}{1000}$ inch of their correct size.

The limitation of this easy method of analysis appears to be when, by the movement of a redundant reaction, a point of inflexion is produced in the model. This causes the model to buckle slightly and to press on the paper, with a consequent increase of sliding

friction. Experimental values for the redundant reactions are then smaller than the calculated values, but points of inflexion are very accurately determined.

Addendum.

Since the above Paper was written the Authors have been enabled, though the courtesy of Messrs. R. T. James & Partners, Aldwych, to apply the method described to a proposed welded steel portal of



Figs. 8.

VARIABLE-SECTION PORTAL.

varying section designed by them for the new City of Westminster Central Depot. The portal is illustrated in *Figs.* 8, and the results of the experiment, which was carried out in precisely the same manner as that for the arch of varying moment of inertia, are given in Table VII.

The experimental influence-line is plotted in *Figs.* 8, and from this

were calculated the values of the horizontal thrusts for two specified loading-conditions, representing the effect of dead loads and live loads respectively. Comparison was then made with the calculations made by the designers, with the following results :—

TABLE VII.—PROPOSED PORTAL FOR NEW CENTRAL DEPOT, CITY OF WESTMINSTER.

Scale 1 inch = 4 feet. Horizontal displacement of H = 15·5.

Distance from A : feet.	Vertical displacement.	H (Mean).	Distance of load from A : feet.	H : tons.	
				Dead load.	Live load.
0	0	0	0	0	0
6·4	1·8	0·12	3·40	0·101	0·146
10·4	2·5	0·17	8·55	0·334	0·441
14·4	3·7	0·24	13·68	0·443	0·715
18·4	4·6	0·30	19·67	0·461	0·894
22·4	5·6	0·36	25·66	0·903	1·243
26·4	6·1	0·41	31·66	0·906	1·449
30·4	6·9	0·46	37·65	0·756	1·470
34·4	7·6	0·49	43·65	1·265	1·720
38·4	8·2	0·54	49·02	1·265	1·720
42·4	8·6	0·56	55·02	0·756	1·470
46·4	8·9	0·57	61·01	0·906	1·449
50·4	8·8		67·01	0·903	1·243
54·4	8·4		73·00	0·461	0·894
58·4	7·9		80·99	0·443	0·715
62·4	7·4		84·12	0·334	0·441
66·4	6·6		89·27	0·101	0·146
70·4	5·7			10·338	16·156
74·4	4·8				
78·4	3·7				
82·4	2·7				
86·4	1·8				
92·8	0				

Total H due to dead load = 10·338 tons
Calculated H due to dead load = 11·058 tons } Difference 6·5 per cent.

Total H due to live load = 16·156 tons
Calculated H due to live load = 16·80 tons } Difference 3·9 per cent.

The time occupied in making the model and in plotting the influence-line was only about 4 hours, and the very close agreement obtained in this case between the results of elaborate calculations and independent experiment inspires considerable confidence in the practical utility of the method.

The Paper is accompanied by eight tracings, from which the Figures in the text have been prepared.

Paper No. 5028.

"Repeated Stresses on Structural Elements."

By Professor FREDERICK CHARLES LEA, O.B.E., D.Sc. (Eng.), M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS.

INTRODUCTION.

DURING recent years the problem of the strains produced in girders by travelling loads has received a good deal of attention, experimentally and theoretically, and from the results of the strain-measurements it has been possible to compare the strains produced by rapidly-moving loads with those produced by the same loads moving slowly, and thus to determine the impact-allowances for certain types of wheel-loads. From the measured strains and the equivalent loads, the stresses in the girder can be obtained by the usual formulas. No attempt has been made by the Bridge Stress Committee,[2] or by other investigators,[3] to determine the relationship of the calculated stresses to the stresses actually occurring at discontinuities or at

[1] Correspondence on this Paper can be accepted up to the 15th March, 1937, and will be published in the October, 1937, Journal.—SEC. INST. C.E.

[2] Report of the Bridge Stress Committee. London, 1928.

[3] Prof. C. E. Inglis, "Oscillations of a Bridge caused by the Passage of a Locomotive," Proc. Roy. Soc. (A), vol. 118 (1928), p. 60.

" " "Impact in Railway Bridges," Minutes of Proceedings Inst. C.E., vol. 234 (1931–32, Part II), p. 358.

R. W. Foxlee and E. H. Greet, "Hammer-Blow Impact on the Main Girders of Railway Bridges," *ibid.*, vol. 237 (1933–34, Part I), p. 239.

W. E. Gelson, "Moving-Load Stresses in Short-Span Railway-Bridges," *ibid.*, vol. 237 (1933–34, Part I), p. 314.

rivet-holes, and thus the actual factors of safety in bridge structures are not known.

The determination of the actual stresses in structural elements can only be effected statically by strain-measurements; in those neighbourhoods where local stresses occur the difficulties of determining the local strains with the necessary precision are considerable. By photo-elastic measurements Professor E. G. Coker, M. Inst. C.E., has investigated many examples of the concentrations of stress when elements are stressed within the elastic range, but, as will be seen later, very considerable differences are found between the stresses so determined and those which apparently cause failure under repeated loading.

It is the object of this Paper to describe certain fundamental experiments that the Author has carried out to determine the safe ranges of stress that can be applied to elements of structures for certain specified numbers of times, and to indicate some of the experiments that are in hand.

The first reference to the effect of moving loads upon structures appears to be that given in a valuable and interesting Report [1] published in 1849. Previous to the work then described, William Fairbairn had carried out long-period tests to determine the effect of continued loading on cast-iron beams; the deflections were measured by means of a microscope.

At a meeting of the British Association in 1860, Fairbairn stated: "Amongst engineers opinions are still much divided upon the question, whether the continuous changes of load which many wrought-iron structures undergo, has any permanent effect upon their ultimate powers of resistance. . . ." The subject was further debated at the British Association meeting in 1861, and in 1864 Fairbairn wrote: [2] "A question of great importance to science and the security of life and property has been left in abeyance for a number of years—namely, to determine by direct experiment to what extent vibratory action, accompanied by alternate severe strains, affects the cohesive force of bodies . . .; the question to be solved is, how long will a body . . . sustain a series of strains produced by impact (or the repeated application of a given force) before it breaks? In the case of bridges and girders, this is a problem on which no reliable information has yet been given. . . ."

Fairbairn carried out, at Manchester, experiments on a girder of

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[1] Report of the Commissioners appointed to inquire into the Application of Iron to Railway Structures. London, 1849.

[2] W. Fairbairn, "Experiments to determine the effect of Impact, Vibratory Action, and long-continued Changes of Load on Wrought-Iron Girders," Phil. Trans. Roy. Soc., vol. 154 (1864), p. 311.

20 feet span, and found that a range of stress of 6·64 tons per square inch from a minimum stress of 0·38 ton per square inch produced no visible effects after 403,000 repetitions, but that a range of stress of 9·3 tons per square inch produced fracture in the tension flange after 5,175 repetitions.

He concluded that wrought-iron girders were not safe "when submitted to violent disturbances with a load equivalent to one-third the weight that would break them. They, however, exhibit wonderful tenacity when subjected to the same treatment with one-fourth the load. . . ." It is strange that so few experiments of the type made by Fairbairn have been carried out, and it is also remarkable that until recent years it does not appear to have been sufficiently recognized how important the discontinuities produced by rivet-holes, by changes of section, and by the surface condition of the material are in determining the range of repeated stress that the material can safely withstand.[1, 2, 3, 4, 5, 6, 7].

Experiments on Mild-Steel "Black" Plates.

The repeated-stress experiments described in this Paper were carried out in the Haigh machine on mild-steel plates with various mean stresses and ranges of stress (in many of the tests the mean stress was adjusted so that the minimum stress was zero). Tests were made :—

(a) on plane specimens,

(b) on plane specimens having holes ranging from 11 thousandths of an inch to ½ inch in diameter,

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1 H. J. Gough, report on "The Present state of Knowledge of Fatigue of Metals," New International Association for the Testing of Materials, Zurich Congress, 1931.

2 ,, "The Fatigue of Metals." London, 1924. (This book contains a comprehensive bibliography.)

3 Prof. F. C. Lea and F. Heywood, "The Failure of some Steel Wires under Repeated Torsional Stresses at Various Mean Stresses, determined from Experiments on Helical Springs," Proc. Inst.Mech.E., vol. I (Jan.–May) 1927, p. 403.

4 Prof. F. C. Lea and R. A. Batey, "The Properties of Cold-Drawn Wires, with particular reference to Repeated Torsional Stresses," Proc. Inst.Mech.E., vol. II (June–Dec.), 1928, p. 865.

5 Prof. F. C. Lea and J. Dick, "Torsional Fatigue Tests on Cold-Drawn Wires," Proc. Inst.Mech.E., vol. 120 (1931), p. 661.

6 R. G. C. Batson and J. Bradley, "Fatigue Strength of Carbon- and Alloy-Steel Plates as used for Laminated Springs," Proc. Inst.Mech.E., vol. 120 (1931), p. 301.

7 Prof. F. C. Lea and J. Dick, Reports (Dec. 1932 and Dec. 1933) of the University of Sheffield Research Department for the Cold-Working of Steel and other Ferrous Metals.

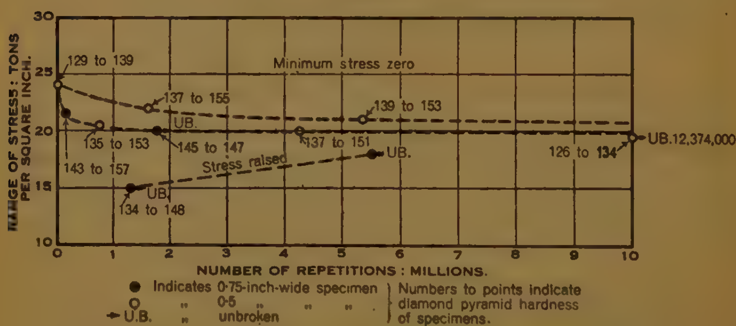
(*c*) on riveted specimens forming simple tie-bars,

(*d*) on bolted specimens in which the grip of the cover-plates was carefully controlled by the nuts.

Tensile Tests and Hardness-Determinations of Materials Used.

Table I (p. 97) shows the static properties and hardness-values of three steels which have been used in the tests. It will be seen by comparing G.335 and G.336 that the strengths of ground and unground specimens, under static stresses, are about the same. Heating the specimens in a salt bath at 890° C. for a few minutes lowered the hardness of specimens G.412 A and G.412 B, and also diminished their tensile strengths. It will be seen that the tensile strengths are, however, almost exactly in proportion to the hardnesses of the

Fig. 1.



TENSILE FATIGUE-TESTS OF MILD STEEL STRIP " A."

specimens. It is clear from these results that the physical properties of thin plates depend to a considerable extent upon their previous history. Many of the tests described have been carried out on plates as delivered, and one series of tests on a series of specimens all of which had the same hardness-number and a tensile strength of about 24·7 tons per square inch. It seems clear from the effect of heat-treatment at 890° C. on plates " C " that some of the plates, as received, had been " cold-worked " by finishing in the rolls below the critical temperature.

Repeated-Stress Tensile Tests, Steel " A."

In Table II (facing p. 98) are shown results of repeated-stress tests on steel " A " for various ranges of stress, the minimum stress in all cases being zero, so that the mean stress was equal to half the range. The results for steel " A " are shown plotted in *Fig. 1 ;* the points for the $\frac{1}{2}$-inch wide plate lie a little higher than for the $\frac{3}{4}$-inch wide

TABLE I.—TENSILE TESTS ON MILD-STEEL STRIP WITH 10-TON BUCKTON TESTING MACHINE.

Test No.	Diamond pyramid hardness :* (load 10 kilograms, time 10 seconds.)			Steel.	Cross section.	Yield-stress : tons per square inch.	Maximum stress : tons per square inch.	Elongation : percentage on 2 inches.
G.84	147	148	143	A	0·75 inch by 0·065 inch	22·6	28·4	30
G.138	130	128	142	B	0·75 ,, 0·061 ,,	19·55	24·3	34·5
G.190	148	134	139	C	0·75 ,, 0·065 ,,	19·95	23·2	39
G.335	150	155	150	C	0·879 ,, 0·065 ,,	23·86	29·6	28
†G.336	146	149	155	C	0·879 ,, 0·053 ,,	22·28	29	31·5
‡G.412 A	144	140	135	C	0·875 ,, 0·066 ,,	13·69	21	43
‡G.412 B	137	142	141	C	0·875 ,, 0·065 ,,	15·87	24·72	40
G.412 C	142	142	141	C	0·874 ,, 0·066 ,,	22·92	28·65	32·5
G.412 D	119	121	121	C	0·875 ,, 0·066 ,,	19·38	23·2	39

All specimens were treated "as received," except G.412 A and G.412 B.

* Three indentations were taken on each specimen. † Faces ground on surface grinder.

‡ Fifteen specimens of steel "C" were clamped together and placed in a salt bath at 890° C. for $1\frac{1}{2}$ hour. The diamond hardnesses were then :—G.412 A : 92·4 92·4 93·6
G.412 B : 108 107 108

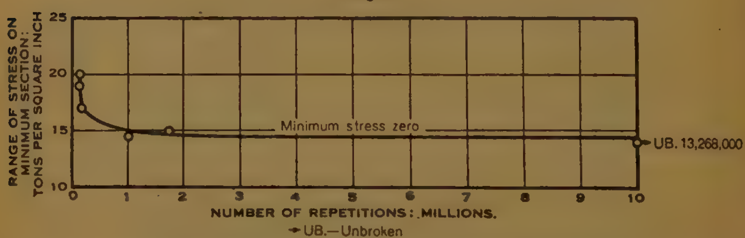
The hardnesses of other specimens similarly heat-treated are shown in Table II.

plate. On *Fig. 1* are also shown the diamond pyramid hardness-numbers obtained from each specimen. It will be seen that for more than 3,000,000 repetitions the range of stress is 20 tons per square inch, so that at zero mean stress the ratio of the safe range to tensile strength is $\frac{20}{28\cdot4} = 0\cdot7$. Many of the specimens cracked, not at the edge of the specimen, but near to the centre. In some cases a slight creep was observed before the crack took place.

Preliminary Tests on Specimens with Small Holes drilled in them.

In *Figs. 2* (facing p. 106) are shown specimens, each drilled with a central $\frac{1}{16}$-inch diameter hole. Specimen 6 withstood 13,268,000 repetitions and remained unbroken. The cracks developing from the hole are clearly visible in specimens 1 to 5, and from the cracks

Fig. 3.



Tensile Fatigue-Tests of Mild-Steel Strip " A "; Specimens 0·75 Inch Wide, with $\frac{1}{16}$-inch Hole through Centre.

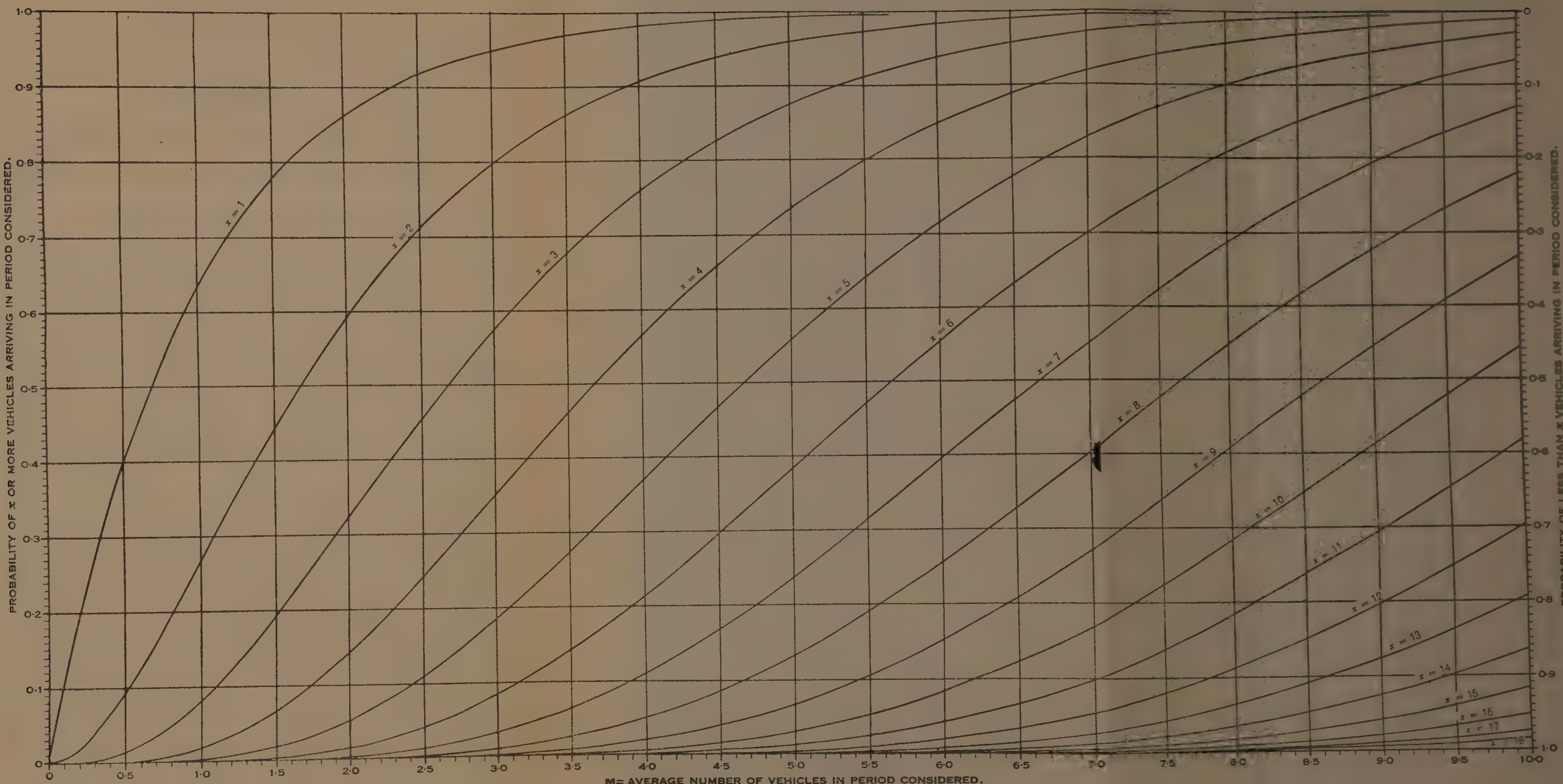
the slip-bands are clearly indicated at an angle of about 45 degrees to the cross section of the specimen.

The results of the repetition tensile tests are shown plotted in *Fig. 3*, from which it will be seen that the fatigue range at zero minimum stress is about 14·4 tons per square inch; or the fatigue range at zero minimum stress for a specimen with a hole $\frac{1}{16}$ inch in diameter is about 0·50 of the tensile breaking strength of the steel and 0·72 of the safe fatigue range without a hole.

Table III (p. 99) gives the results of only three tests on specimens of steel " A " with holes from 11 to 12 thousandths of an inch in diameter, drilled at the centre of the specimen. The cracks in each specimen that broke started from the small hole. The results of the tests on steel " A " with a small hole were not very conclusive, as all the specimens broke, but one specimen, at a range of stress of 20 tons per square inch, with zero minimum stress, withstood nearly 3,000,000 repetitions. Undrilled specimens of about the same hardness,

ROAD TRAFFIC CONSIDERED AS A RANDOM SERIES.

FIG. 3.



Note: Normal (Random) traffic distribution is assumed.

DIAGRAM SHOWING PROBABILITY THAT THE NUMBER OF VEHICLES ARRIVING IN A GIVEN TIME WILL BE *X* OR MORE (OR LESS THAN *X*).

The Institution of Civil Engineers. Journal. November, 1936.

W. F. ADAMS.

WATERLOW & SONS LIMITED, LATE THOS. KELL & SON, LONDON.

TABLE III.—REPEATED-STRESS TESTS ON MILD-STEEL STRIP IN THE HAIGH MACHINE.

SPECIMENS WITH SMALL HOLES OF VARIOUS DIAMETERS.

Test No.	Diamond pyramid hardness: * (load 10 kilograms, time 10 seconds.)	Range of stress: † tons per square inch.	Mean stress: † tons per square inch.	Repetitions.	Size of hole: inch.	Width of specimen: inch.	Thickness of specimen: inch.	Remarks.
				Steel "A."				
G.118	146 150 157	20	10	2,954,000	0·011	0·5	0·069	Broken.
G.126	140 146 138	20·9	10·45	78,000	0·012	0·5	0·069	,, Creep.
G.131	140 143 141	18·5	9·25	630,000	0·012	0·5	0·065	,,
				Steel "B."				
G.147	145 150 143	18	9	202,000	0·012	0·5	0·060	,, ,,
G.148	122 124 117	16	8	3,086,000	0·012	0·5	0·060	,,
G.149	162 153 160	15·5	7·75	2,610,000	0·013	0·5	0·060	,,
G.150	130 131 129	14.5	7·25	1,270,000	0·017	0·5	0·060	,,
G.167	133 137 133	14	7	12,416,000	0·013	0·75	0·060	Unbroken.
G.172	127 122 124	15	7·5	6,060,000	0·014	0·75	0·060	,,
G.174	126 134 135	16	8	11,144,000	0·015	0·75	0·060	,,
G.175	124 131 134	17	8·5	11,026,000	0·012	0·75	0·060	,, Hole $\frac{1}{16}$ inch out of centre.
G.231	127 127 127	18	9	318,000	0·015	0·75	0·060	Broken.
G.232	137 137 127	16	8	8,710,000	0·013	0·75	0·060	Unbroken.
G.233	130 137 143	17	8·5	1,884,000	0·014	0·75	0·060	Broken.

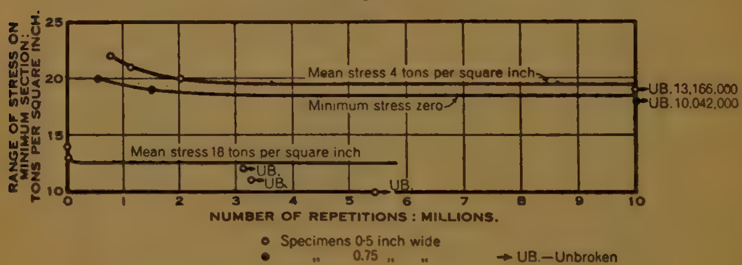
* Three indentations were taken on each specimen. † Stresses calculated on the minimum section.

Fig. 1 (p. 96), failed at about the same range. Another specimen at a range of 18·5 tons per square inch broke, however, after 630,000 repetitions. These tests showed that drilling small holes in the specimens introduces some uncertainty in the range which would finally cause fracture. Further, the type of failure of plane specimens without a hole was very similar to that of those with a small hole, which suggests that the crack in the plane specimens most probably commenced at a surface defect where there was some concentration of stress.

Repeated-Stress Tests, Steel "B."

Tests on steel "B" were now made at various mean stresses. For the undrilled plates the results are plotted in *Fig. 4*. At zero mean stress the fatigue range is about 18·5 tons per square inch, but this is probably lower than it should be, owing to slight buckling.

Fig. 4.

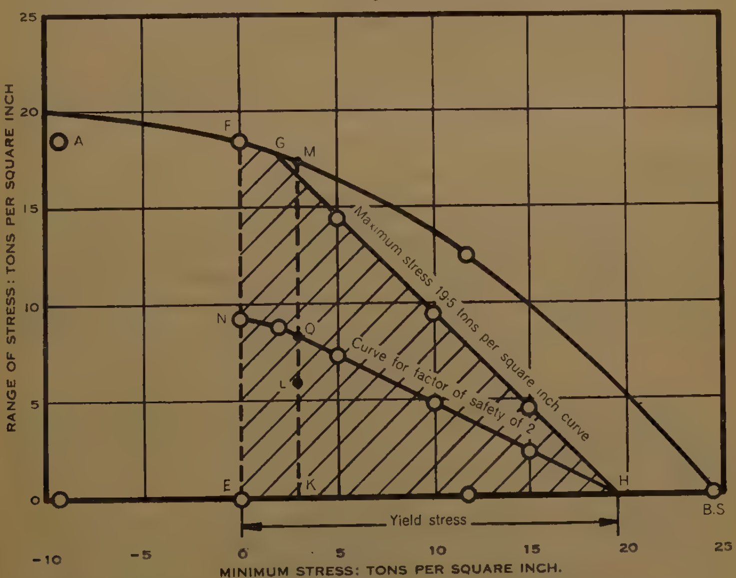


TENSILE FATIGUE-TESTS OF MILD-STEEL STRIP "B."

At a mean stress of 18 tons per square inch and a range of 14 tons per square inch, fracture took place after 4,000 repetitions; but with a range of stress of 10 tons per square inch the specimen withstood 5,460,000 repetitions without fracture, and the safe range of stress was about 12·5 tons per square inch. In *Fig. 5*, the safe range of stress is plotted on a base of minimum stress. The plottings indicate that the safe range obtained at zero mean stress is apparently lower than the true safe range. It was at first intended to test specimens in tension only, as compressive stresses would tend to buckle the thin specimens. The tests at zero mean stress confirmed this fear, and the point marked A on *Fig. 5* is below the true curve. It will be seen that the maximum stress, when the minimum is about 12 tons per square inch and the mean stress is 18 tons per square inch, is 24·25 tons per square inch and is, therefore, only a little less than the tensile strength of the material shown in Table I (p. 97). As might be expected, creep took place in all the specimens tested at a mean stress of 18 tons per square inch. At a mean stress of 4 tons per square

inch and a range of 19 tons per square inch, specimens were unbroken at more than 13,000,000 repetitions. The maximum stress was then 13·5 tons per square inch (tension) and the minimum stress — 5·5 tons per square inch (compression). From *Fig. 5* it is possible approximately to interpolate the safe range of stress for any specified minimum or mean stress. For example, let the dead-load stress on an element be 3 tons per square inch, EK; the safe range is then KM. If, therefore, the stress produced by the moving load and other causes is KL tons per square inch, the factor of safety is $\frac{KM}{KL}$.

Fig. 5.



Safe Range of Stress for Mild-Steel Strip "B," Undrilled.

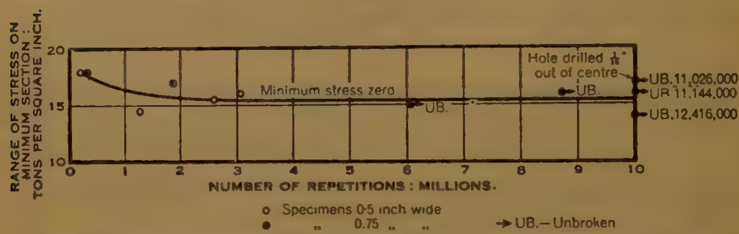
The curve of range of stress on a base of minimum stress is very approximately a parabola; its equation is $R = 20 - \frac{(m + 10)^2}{59}$, where R denotes the safe range and m the minimum stress. Under direct-stress tests creep takes place at stresses above the yield-stress of the material; if no alteration of dimensions is desirable the maximum stress should, strictly speaking, be less than the limit of proportionality, and if no appreciable alteration of dimensions is desirable the maximum stress should be less than the yield-stress. Assuming this, as an example, to be 19·5 tons per square inch, the hatched area EFGHK (*Fig. 5*) is then the area within which the design of tension elements must fall if neither the safe range nor

the yield-stress is to be exceeded. Assuming a factor of safety of 2, the safe range of stress, for this material, for any specified minimum dead-load stress is given by the curve ENQH.

Further Tests of Specimens with Holes Drilled in them.

Table III (p. 99) and *Fig. 6* show results obtained from specimens of steel "B" with holes ranging from 12 to 17 thousandths of an inch

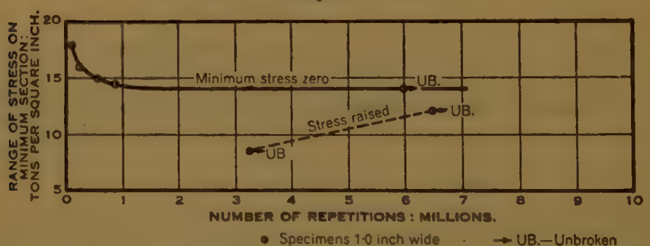
Fig. 6.



TENSILE FATIGUE-TESTS OF MILD-STEEL STRIP "B," WITH SMALL HOLE THROUGH CENTRE.

in diameter drilled at the centre of the specimen; one specimen, G.175, had the hole $\frac{1}{16}$ inch out of centre. An examination of the Table and of *Fig. 6* shows that the drilling of the small holes, as already indicated for steel "A," introduces an element of uncertainty, but

Fig. 7.



TENSILE FATIGUE-TESTS OF MILD-STEEL STRIP "B," WITH $\frac{1}{2}$-INCH HOLE THROUGH CENTRE.

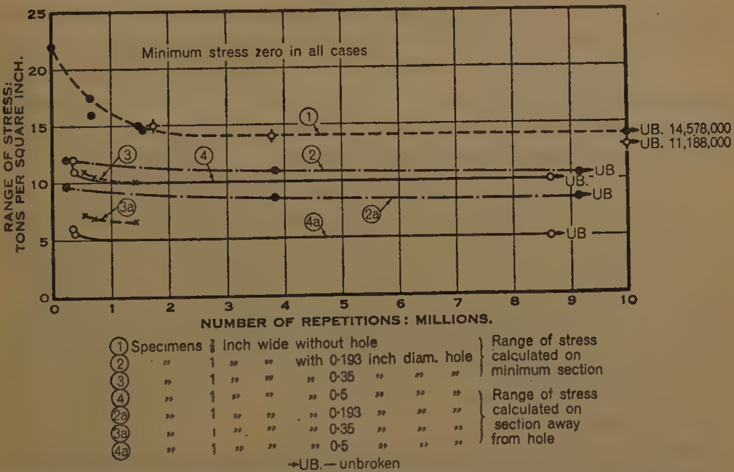
for zero minimum stress the safe range for 3,000,000 stress-repetitions is not far removed from 15 to 17 tons per square inch; the hole being out of centre did not appreciably affect the safe range of stress.

Fig. 7 shows results of tests from 1-inch wide specimens with a $\frac{1}{2}$-inch hole drilled at the centre of the specimen. All tests were taken at zero minimum stress, and all specimens that broke cracked on the horizontal diameter at the hole boundary. At zero minimum stress the safe range is 14 tons per square inch, and is thus less than the safe range with the small hole and with the $\frac{1}{16}$-inch diameter hole.

Tests of Heat-Treated Plates, with Holes Drilled in them.

A further series of tests was now made upon steel " C." A number of plates were carefully heat-treated at 890° C. to ensure as far as possible uniformity of hardness. The hardnesses of nine plates, taking three impressions on each plate, ranged only from 86·4 to 93·2, and of twenty-seven impressions only five gave hardnesses below 90. The tensile strength of the plate after heat-treatment was 21 tons per square inch, and the elongation on 2 inches was 43 per cent. Holes 0·5 inch, 0·35 inch and 0·193 inch diameter were drilled in specimens 1 inch wide. Stresses were calculated on

Fig. 8.



Tensile Fatigue-Tests of Heat-treated " Black " Mild-Steel Strip " C."

the sectional area of the plate through the centre of the hole and also on the section away from the hole.

The results of the tests are shown plotted in *Fig. 8*, the results of tests without the hole being also shown. It will be seen that the size of the hole makes very little difference to the fatigue range, which for 10^7 repetitions is about 10 tons per square inch (about two-thirds of the fatigue range for the undrilled plate, and a little less than one-half of the breaking strength of the plate). As will be seen later, these comparative results do not agree with what might be expected from strictly theoretical analyses or from photo-elastic determinations; an explanation is offered on p. 104.

Tests were also made on specimens of this plate which had had the surfaces carefully polished. The fatigue range was higher than for

the black plates; the plates showed a definite diminution in thickness in places, but cracked at sections where there was no such indication.

Comparison of Results of Repeated-Stress Tests with Theoretical Investigations and Photo-Elastic Tests.

The distribution of stress in a plate under tension pierced by a hole having its centre on the axis of the specimen has been investigated theoretically and experimentally. As shown by Professors E. G. Coker and L. N. G. Filon, when the plate is infinitely wide and the hole is small a solution can be found without difficulty.[1] Professor G. B. Jeffery [2] has given a solution for a semi-infinite plate with one circular hole and, as pointed out by Mr. R. C. J. Howland,[3] the method may be applied to the case of an infinite plate pierced by two holes. Professor Coker and Mr. T. Fukuda have shown [4] that in a tension member loaded through a pin piercing a hole of diameter 0·1275 times the width of the specimen on the axis of the specimen, the maximum stress at the hole is six times the mean stress on the specimen. Mr. Howland [5] has also investigated the stress-distribution around the hole in a plate bounded by two parallel edges for various ratios of the diameter of hole to the width of the plate. Professor Coker [6] has also investigated the stress-distribution existing at the boundaries of the hole by photo-elastic measurement. In the case of a hole $\frac{1}{4}$ inch in diameter in a plate 1 inch wide, he found that for a pull corresponding to a mean stress T of 569 lbs. per square inch the measured stress at the hole boundary was 1,720 lbs. per square inch.

Table IV shows comparisons [7] of the stresses obtained experimentally and by calculation in the neighbourhood of a hole whose diameter is half the width of the plate. The angle θ defines the position of a point on the edge of the hole from a diameter perpendicular to the axis of the specimen; $\frac{\widehat{\theta\theta}}{T}$ shows the ratio of the

[1] E. G. Coker and L. N. G. Filon, "Photo-Elasticity," p. 483. Cambridge, 1931. (This volume contains a comprehensive bibliography.)

[2] G. B. Jeffery, "Plane Stress and Plane Strain in Bipolar Co-ordinates," Phil. Trans. Roy. Soc. (A), vol. 221 (1920), p. 265.

[3] R. C. J. Howland, "On the Stresses in the Neighbourhood of a Circular Hole in a Strip Under Tension," Phil. Trans. Roy. Soc. (A), vol. 229 (1929), p. 49.

[4] E. G. Coker and L. N. G. Filon, "Photo-Elasticity," p. 524. Cambridge, 1931.

[5] Howland, *loc. cit.*

[6] Coker and Filon, *loc. cit.*

[7] Taken from the Paper by Mr. R. C. J. Howland (see footnote (3), above).

stress at the hole to the mean stress in the plate. It will be seen that in this particular case the maximum ratio of $\frac{\widehat{\theta\theta}}{T}$ found by experiment is 3·53 and by calculation 4·32. At a point near to the hole, where ρ is 0·55, the ratio by calculation is 3·34; the agreement between experiment and theory may therefore be considered good. When a large hole was made in a plate Professor Coker found [1] that the maximum stress at the boundary of the hole was about twice the mean stress on the reduced area. On reference to the results

TABLE IV.—COMPARISON OF THEORETICAL RESULTS OBTAINED BY MR. R. C. J. HOWLAND WITH THOSE OBTAINED EXPERIMENTALLY BY PROFESSOR E. G. COKER AND MESSRS. CHAKKO AND SATAKE, USING OPTICAL METHODS.*

θ: degrees.	Stress at rim of hole found by optical methods (ratio of diameter of hole to width of plate $\lambda = 0{\cdot}5$).		Stress at rim of hole $\lambda = 0{\cdot}5$ and on a neighbouring circle ($\rho = 0{\cdot}55$†), by calculation.	
	$\widehat{\theta\theta}$	$\widehat{\theta\theta}/T$	$\widehat{\theta\theta}/T$ for $\rho = 0{\cdot}5$	$\widehat{\theta\theta}/T$ for $\rho = 0{\cdot}55$
0	1,960	3·53	4·32	3·34
15	1,800	3·24	3·72	2·94
30	1,190	2·12	2·32	1·94
45	490	0·88	0·77	0·78
60	—280	—0·50	—0·51	—0·22
75	—600	—1·08	—1·32	—0·84
90	—700	1·26	—1·58	—1·06

* See Phil. Trans. Roy. Soc. (A), vol. 229 (1930), p. 75.

† ρ denotes the ratio of a radius at which the stress is measured to the half-width of the plate.

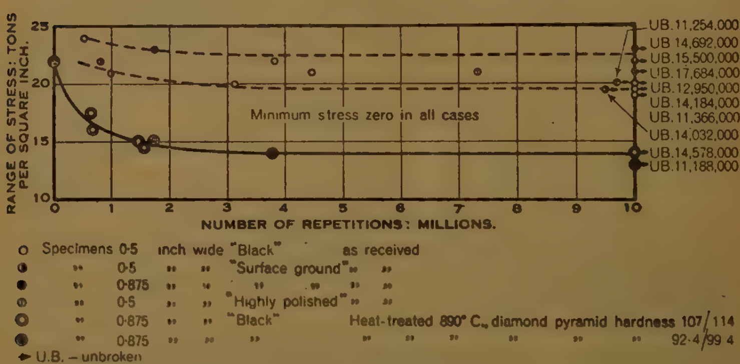
given in the text, it will be seen that the safe range of stress, reckoned on the diminished area at zero mean stress, was, for the plate 1 inch wide with a ½-inch hole, about 14 tons per square inch. The mean stress well removed from the hole was therefore about 7 tons per square inch, or a little more than one-third of the stress that caused failure of the undrilled plate. Again, the safe range of stress for the plates with small holes is not very different from that for the solid plate. If it be assumed that there was a stress-concentration of 3, it will appear that the stress at failure was much higher than in the undrilled plate. The fracture in the undrilled plate probably starts at a small discontinuity, which itself is causing a stress-concentration similar to that at a small hole.

As a partial explanation of the differences between the safe ranges of stress and the theoretical results given by Professor Coker and Mr. Howland, it may be suggested that the plastic deformations in

[1] E. G. Coker and L. N. G. Filon, "Photo-Elasticity," p. 486. Cambridge, 1931.

the neighbourhood of points of maximum stress harden the material, and also that they allow of a more uniform distribution of stress than indicated by the strict elastic conditions. On reference to *Fig. 5* (p. 101) it will be seen that maximum stresses approximating to the ultimate stress of a material may be applied to a specimen, provided that the range of stress is not greater than a certain amount, without causing fracture. In such cases, however, permanent measurable deformation of the specimen takes place. It is not difficult, therefore, to visualize very small plastic movements, hardly measurable, taking place in the neighbourhood of points of concentration of stress, and hardening the material, so that the plastic movements cease.[1] The Author has shown [2] that specimens of austenitic steel, a material very susceptible to cold work, can be very considerably hardened by

Fig. 9.



TENSILE FATIGUE-TESTS ON PLAIN SPECIMENS OF MILD-STEEL STRIP "C."

subjecting them to many millions of stress-repetitions without producing fracture.

In order to test whether, in the mild steels studied, the discontinuities due to the black surfaces were important, certain specimens of steel "C" were ground so that their surfaces were bright; others were highly polished. The results of these tests are shown in Table II (facing p. 98) and in *Fig. 9*.* Comparing the results from the ground and polished specimens with those obtained from the black specimens, it will be seen that there is a definite indication that the

[1] Pyramid diamond hardness-tests were taken at a great number of points along the specimen, across the specimen coincident with the diameter of the hole, and around the hole. There was definite evidence of hardening.

[2] Prof. F. C. Lea, "Strength of Materials as affected by Discontinuities and Surface Conditions." Journal Society of Glass Technology, vol. 16 (1932), p. 182.

* See also *Fig. 8*.

Figs. 2.

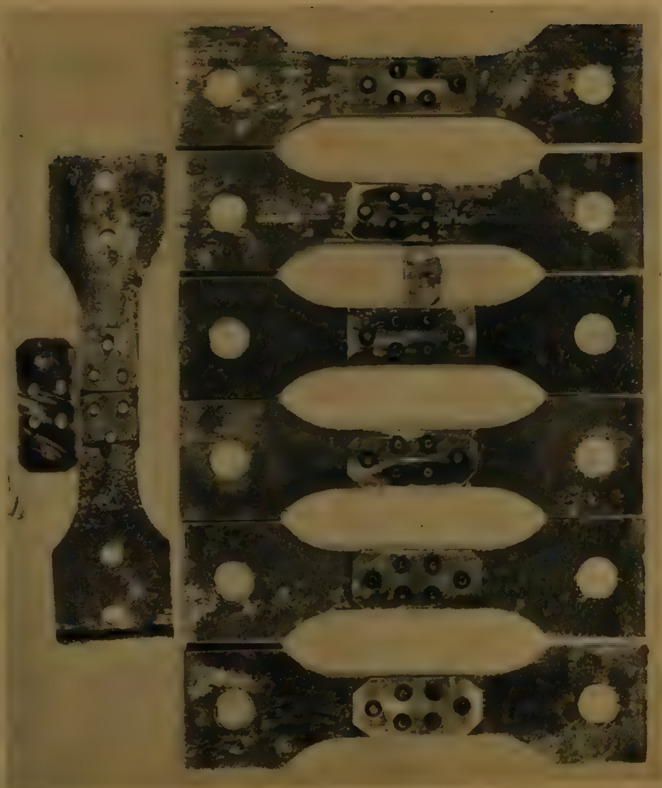
Specimen.	*Range of Stress.*	*Result.*
G.88	0—20 tons per square inch ...	Failed after 150,000 repetitions.
G.100	0—19 ,, ,, ,, ,, ...	Failed after 148,000 repetitions.
G.101	0—17 ,, ,, ,, ,, ...	Failed after 188,000 repetitions.
G.102	0—15 ,, ,, ,, ,, ...	Failed after 1,748,000 repetitions.
G.109	0—14·5 ,, ,, ,, ,, ...	Failed after 1,020,000 repetitions.
G.108	0—14 ,, ,, ,, ,, ...	Unbroken after 13,268,000 repetitions.

1 2 3 4 5 6

MILD-STEEL STRIP SPECIMENS WITH $\frac{1}{16}$" DIAMETER HOLE.

Figs. 11.

Specimen.	*Range of Stress.*	*Result.*
G.91	0—18 tons per square inch ...	Failed after 12,000 repetitions.
G.92	0—15 ,, ,, ,, ,, ...	Failed after 70,000 repetitions.
G.93	0—10 ,, ,, ,, ,, ...	Failed after 270,000 repetitions.
G.97	0—9 ,, ,, ,, ,, ...	Failed after 700,000 repetitions.
G.111	0—8·5 ,, ,, ,, ,, ...	Failed after 524,000 repetitions.
G.94	0—8 ,, ,, ,, ,, ...	Unbroken after 14,504,000 repetitions.
	0—9 ,, ,, ,, ,, ...	Unbroken after 2,506,000 repetitions.



G. 105. Tensile Test. Maximum Stress 26·9 tons per square inch.

MILD-STEEL STRIP, RIVETED SPECIMENS.

grinding and polishing of the specimen has increased the safe range, but only to a comparatively small extent. The black surfaces of hard quenched and tempered steels affect the safe range of stress to a very marked extent.[1] It can be said, therefore, that the drilling of holes in mild-steel structural members subjected to repeated stresses does not reduce the safe range of stress, as compared with the range for the undrilled plate, to the extent that might be inferred, without fuller consideration, from the theoretical analysis of Mr. Howland or the experimental work of Professor Coker.

Failure of Riveted Specimens under Repeated Stresses; Effect of Tightness of Grip of Cover-Plates.

In structural work holes will generally be filled with rivets or bolts. Professor Coker has determined the elastic stress-distributions in the neighbourhood of a rivet fitting a central hole in a tension member. The maximum stress at the rivet-hole boundary in a plate 1 inch wide having a rivet 0·125 inch in diameter was a little greater than twice the mean stress on the plate away from the hole. When the load was applied to a plate 18 inches by 6 inches by 0·17 inch through a pin, the tensile stress, as shown by the photo-elastic measurements, at the boundary of the hole was more than six times the mean stress on the plate. The following tests indicate the effect of stress-concentrations in the neighbourhood of rivet-holes, and show very clearly the effect of the "grip" of riveted cover-plates in causing an increase in the safe range of repeated stress over and above that for bolted joints, but again they show that the safe range of stress is very much greater than one-sixth of that for the undrilled plate.

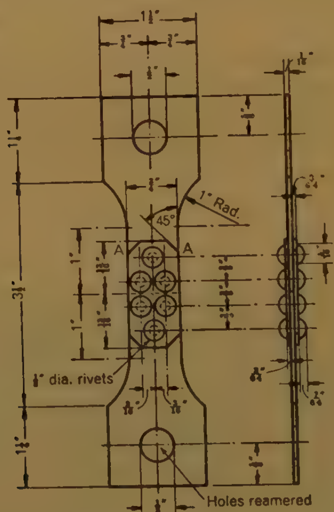
Fig. 10 (p. 108) is a drawing of a riveted element for repeated-stress and static tests, made of steel "A" having a tensile strength of 28·4 tons per square inch. Photographs of the static and repeated-stress specimens are shown in *Figs. 11*. Specimen G.105 was broken in the testing machine; it failed on the section of the plate across the top rivet-hole at a stress of 26·9 tons per square inch on the net section, which is 1·5 ton per square inch less than the stress at which the plate specimens failed (Table I, p. 97). It will be noticed on reference to *Fig. 10* that the cover-plates were $\frac{3}{64}$ inch thick, and that there were three rivets only on each side of the joint. The rivets were inserted hot and the riveting done by a quick-acting hand-press. In Table V (p. 109) are given the results of the repeated-stress tests at zero minimum stress. It will be seen that the safe range of stress is

[1] See footnotes 1, 2, 3, 4, 5, 6, 7 on p. 104.

about 8 tons per square inch, or less than one-third of the static breaking stress on the same riveted joint, and is about 0·42 of the safe range of stress of the plate without a hole in it. The calculated bearing stresses on the rivets are given in Table V.

Removing the cover-plate after the specimen had fractured showed that the fracture was across the horizontal diameter of the top rivet-hole. All the riveted specimens which broke under the repeated stresses cracked in the same way. As already noticed, under the tensile test the specimen G.105 (*Figs. 11*) also fractured across the top rivet-hole.

Fig. 10.



Scale one-third full size.

RIVETED-JOINT SPECIMEN; TYPE "O."

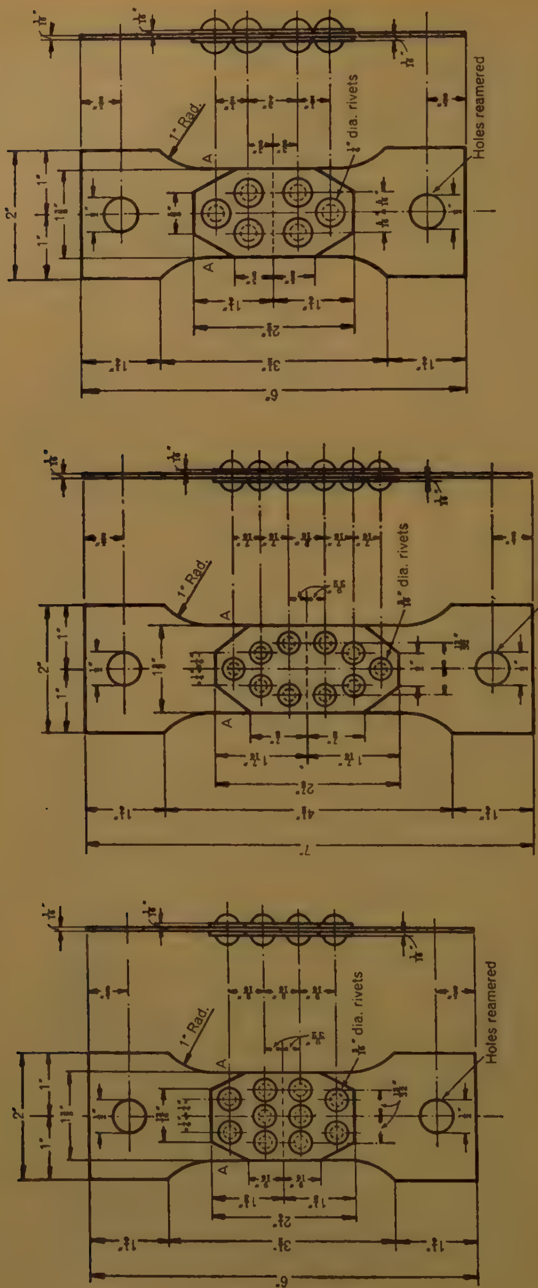
Figs. 12 shows three designs of riveted joints made of plate "B." It will be seen that the cover-plates in these joints are $\frac{1}{16}$ inch thick and that five rivets have been used on each side of the joint instead of three. The rivet-diameters are also greater than in the joint of *Fig. 10*. The results of the repeated-stress tests at zero minimum stress are plotted in *Fig. 13*. The bearing stress for a specimen which withstood $9{\cdot}84 \times 10^6$ repetitions without fracture was higher than for the two specimens G.94 and G.111, Table V. The differences in the results obtained from the various joints and shown in *Fig. 13* did not appear, therefore, to depend upon the differences of bearing stress, but rather depended upon the tightness with which the cover-plates were riveted to the specimen. It was therefore thought desirable to test a joint similar to joint R but with bolts inserted

TABLE V.—REPEATED-STRESS TESTS ON RIVETED SPECIMENS OF MILD-STEEL STRIP IN THE HAIGH MACHINE.
(Joint Type O.)
Steel "A."

Test No.	Diamond pyramid hardness: * (load 10 kilograms, time 10 seconds.)	Range of stress: tons per square inch.	Mean stress: tons per square inch.	Repetitions.	Remarks.	Maximum bearing stress f_b: tons per square inch.
G.91	141 140 140	18	9	12,000	Broken	30
G.92	150 143 139	15	7·5	70,000	,,	25
G.93	145 152 151	10	5	270,000	,,	16·66
G.94	140 145 135	8	4	14,504,000	Unbroken Stress raised	13.33
		9	4·5	2,506,000	Unbroken	15
G.97	145 149 148	9	4·5	700,000	Broken	15
G.111	139 135 146	8·5	4·25	524,000	,,	14·16
†G.105	160 164 170	—	—	—	—	—

* Three indentations were taken on each specimen.
† Tensile test in 10-ton Buckton machine. Maximum stress = 26·9 tons per square inch.
Stresses calculated on section at AA (*Fig. 10*).

Figs. 12.



Type "R."

Scale one-third full size.

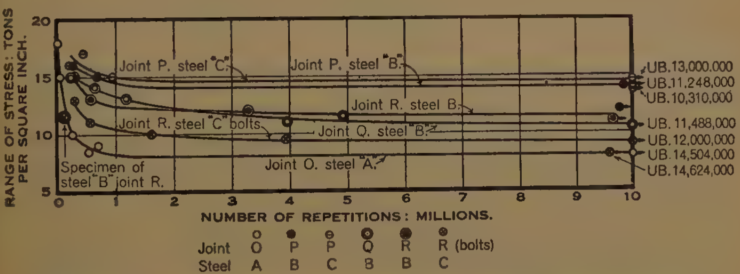
Type "Q."

Riveted-Joint Specimens.

Type "P."

instead of rivets. The holes in the plates were very carefully drilled and the bolts made a good fit in the holes. The heads of the bolts were slotted to take a screw-driver. The joints were apparently reasonably tight, but, as will be seen, there could not be any very considerable pressure between the cover-plates and the plate under test. The results of the tests on the bolted joint are plotted on *Fig.13*. The safe range of stress at zero minimum stress is about 9 tons per square inch. The same joint, riveted, gave a safe range of stress at zero minimum stress of about 11·5 tons per sq. inch. It will be seen on reference to pp. 94–95, that an examination of Fairbairn's experiments on a mild-steel girder shows that a stress of 9·3 tons per square inch produced a fracture across a rivet-hole in a little

Fig. 13.



TENSILE FATIGUE-TESTS ON RIVETED SPECIMENS OF MILD-STEEL STRIP, "A," "B," AND "C."

over 5,175 repetitions of stress. The results of these tests on comparatively small specimens also indicate very definitely that repeated stresses may produce fractures in the neighbourhood of rivet-holes at considerably smaller stresses than those required to produce fracture in the original plate. The Author has designed two machines, one for testing plates with or without holes in them under cycles of bending stress, and the other for testing small riveted or welded structures under cycles of bending stress: if the results of tests from these machines prove to be of interest, it is hoped that they may indicate the desirabilty of experiments on a still larger scale.[1]

WIRE FOR SUSPENSION-BRIDGES; REPEATED-STRESS TESTS ON COLD-DRAWN WIRES.

There has recently been a considerable development of long-span suspension-bridges, particularly in the United States of America.

[1] See p. 95.

The cables consist of wire ropes, and it is of importance, in connection with such developments, that the ranges of stress which the wires composing the ropes are able to withstand indefinitely should be known. The Author has for a number of years been carrying out repeated-stress experiments on wires, and he has shown definitely that the safe ranges of stress which cold-drawn wires will withstand are very much smaller than might be expected from the tensile strengths of the wires. The results of these researches have been published in a number of Papers.[1]

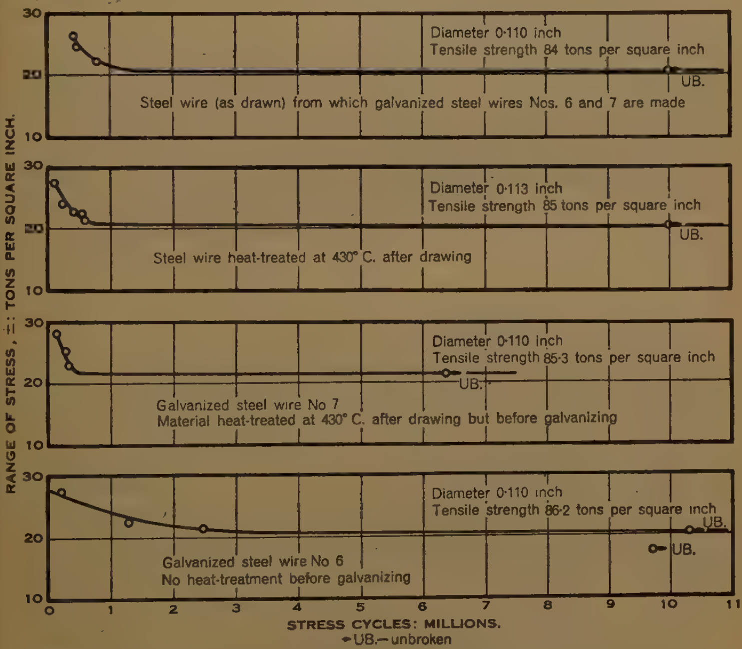
Experiments have been carried out on the wires under repeated torsional stresses, repeated bending stresses, and repeated direct stresses. The work has shown very clearly that the surface-condition of the wires has an important effect upon the safe range of repeated stress, and it has also been shown that during cold-working some particular condition is set up in the wire which makes the safe range of repeated stresses very much less than that obtained from a machined and ground material of the same composition and having the same tensile strength produced by hardening and tempering. For example, a wire of 0·60-per cent. carbon steel, which had been hardened, tempered, and ground after tempering, had a tensile strength of 107·5 tons per square inch ; the safe fatigue-range at zero mean stress under torsional stresses was $\pm$ 27·5 tons per square inch. The same steel in the cold-drawn condition, and having a tensile strength of 89 tons per square inch, only gave a safe fatigue-range of about $\pm$ 15 tons per square inch. A wire having a percentage content of 0·65 carbon, 0·42 manganese, 0·12 sulphur and 0·14 phosphorus was drawn to have a tensile strength of 89·9 tons per square inch. The surface hardness was about 400 and the hardness at the centre 450. The safe range of repeated torsional stress was only $\pm$ 13·6 tons per square inch. A similar wire was made from a hot-drawn rod which was ground before drawing the wire. The decarburized surface of the wire was thus removed, and the safe range of repeated torsional stress was found to be $\pm$ 16·3 tons per square inch. Many wires have been tested having a tensile strength of well over 100 tons per square inch, which at zero mean stress had a safe range of torsional repeated stress less than $\pm$ 15 tons per square inch. The best result that has been obtained from a drawn wire, the whole of the decarburized surface of the hot rod being removed before drawing, has been a range under torsional stress of $\pm$ 20 tons per square inch at zero mean stress.

[1] See footnotes 1, 2, 3, 4, 5, 6, 7 on p. 104.

Repeated Bending-Stress and Direct-Stress Tests on Cold-Drawn Wires.

Repeated bending-stress tests on many cold-drawn wires have been carried out in a machine designed by the Author [1] in which the whole length of the specimen is subjected to the same bending moment, and also in the Haigh machine. *Figs.* 14 show typical

Figs. 14.



Repeated Bending Tests on 0·55-per-cent. Carbon-Steel Wire.

results. The safe range of bending-stress at zero mean stress is about ± 20 tons per square inch, the total range being one-half of the tensile strength of the wire. Table VI shows results kindly obtained for the

Table VI.

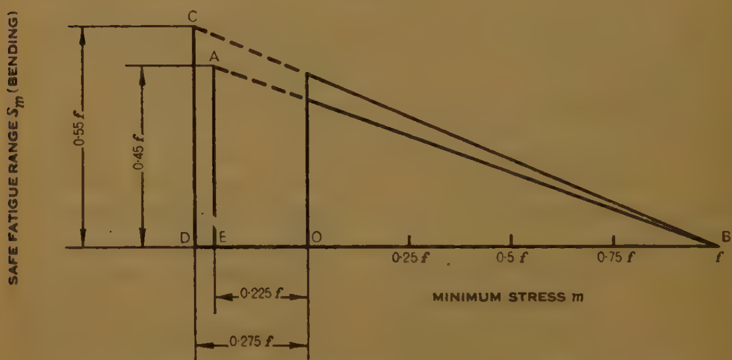
Range of stress: tons per square inch.	No. of reversals of stress: millions.	Remarks.
± 27·5	0·338	Broken
± 26·0	0·545	,,
± 25·0	0·645	,,
± 24·0	0·782	,,
± 23·0	1·167	,,
± 22·0	15·93	Unbroken

[1] Prof. F. C. Lea, "The Effect of Repetition Stresses on Materials," *Engineering*, vol. 115 (1923, Part I), pp. 217, 252.

Author by Professor B. P. Haigh, M. Inst. C.E., and Mr. T. S. Robertson, on the elegant repeated-stress machine of their design in which the test specimen revolves as an Euler column. The wire used was 0·1 inch in diameter, containing 0·6 per cent. of carbon, with an ultimate tensile strength of 82·2 tons per square inch and a reduction of area of 42 per cent. The fatigue-range at zero mean stress was found to be about 44 tons per square inch for 10^6 repetitions, being thus about 0·535 of the tensile strength.

It will be seen that in the cases given the total range of repeated bending-stress is a little greater than 40 tons per square inch, and varies from about 0·47 to 0·535 of the ultimate breaking-strength of the wire. Other tests have shown that the ratio may be less than 0·4. Attempts are now being made to draw wires which shall have a much larger ratio of the safe fatigue-range to the tensile strength

Fig. 15.



Safe Ranges of Bending-Stress for Cold-Drawn Wires.

than 0·535. Incidentally, it should be mentioned that the effect of corrosion on the safe range of repeated stress of steels, other than non-corroding steels, is very serious, but galvanizing is an almost perfect protection, so long as the surface is not damaged.

It is not out of place to draw attention to the difficulty of holding cold-drawn wires in either the torsional repeated-stress machine or in the bending machine so as to ensure fracture away from the gripped ends. This difficulty has been overcome in the Author's laboratory, and there is no such difficulty in the Haigh-Robertson "column bending machine" referred to above. Any discontinuity at a grip leads to stress-concentration and fracture at a low apparent stress. Grips on winding-ropes and on overhead lines are not infrequently responsible for early failure.

Suspension-bridge wires in service will clearly be subjected to dead-load stresses and to repeated stresses due to the travelling loads.

Fig. 15 shows the safe ranges of bending-stress for cold-drawn wires, plotted on a base of minimum stress, interpolated from repeated torsional and bending-stress tests on many classes of cold-drawn wire. The safe range of stress S, at any mean stress M, of a hard steel or a steel wire having an ultimate tensile stress f, can be expressed as $S = S_0(1 - \frac{M}{f})$ where S_0 denotes the safe range of stress at zero mean stress.[1]

In *Fig. 15*, which shows what the Author considers to be the limits of safe ranges of bending and direct stress for various minimum-stress values, the safe range of stress S_m is plotted against minimum stress m. The equation of the straight line AB is :—

$$S_m = \frac{0{\cdot}45f\,(f - m)}{1{\cdot}225f} = 0{\cdot}367(f - m),$$

and for the line CB is :—

$$S_m = \frac{0{\cdot}55f\,(f - m)}{1{\cdot}275f} = 0{\cdot}43(f - m).$$

Tests on wires having an ultimate tensile breaking strength of more than 100 tons per square inch show that, unless very special precautions are taken with the manufacture, the fatigue-range at zero mean stress may be less than $0{\cdot}5f$; it appears, therefore, that the fatigue-range at any mean stress m can safely be assumed to be (from 0·36 to 0·43) $(f - m)$, or approximately $0{\cdot}40(f - m)$. Probably the maximum that at present may be possible is $S_m = 0{\cdot}5(f - m)$. Denoting the factor of safety by S, the working live-load stress may be reasonably taken as :—

$$p = \frac{0{\cdot}40(f - m)}{S}.$$

Assuming the factor of safety S to be 3, f to be 80 tons per square inch, and m to be 15 tons per square inch, the working stress $p = 8{\cdot}6$ tons per square inch.

When the dead-load stress is high, therefore, cold-drawn wires are only capable of resisting a comparatively small range of repeated stress. Furthermore, at clamps the safe range of repeated stress, due to any bending that may come upon the wires from a cross wind or mechanical constraint, will be diminished. It should also be recognized that if a wire of diameter d has to be bent from the straight into any radius r (for example, round a pulley) there is superimposed on the direct stresses a bending stress

$$f_m = \frac{Ed}{2r}.$$

[1] See footnotes 3, 4, 5, 7, p. 104.

If the wires in the rope are in any way prevented from sliding relative to each other when passing over a pulley or round a radius, the bending stress may be greater than that given by the above equation. Further, if the rope be supposed to turn through 180 degrees between consecutive passes round the pulley, then the total range of repeated stress will be $2f_m$.

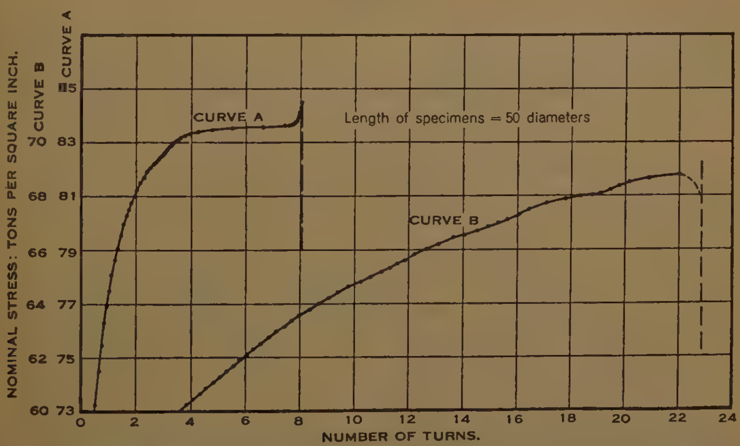
In addition to the ability to resist tensile or repeated stresses, other properties are required by bridge wires. In 1929 it was reported [1] that: "The Mount Hope Bridge at Bristol, R.I., U.S.A., has a span of 1,200 feet with two cables 11 inches in diameter. Each cable was built up of seven strands, each containing 350 No. 6 gauge galvanized wires . . ." which had been specially heat-treated. Further particulars of the wires and the specifications are given. The whole of the wire met the requirements of the tests satisfactorily. "As the work of erection approached completion, however, an inspector detected three broken wires in the neighbourhood of one of the anchorage shoes, and a subsequent examination disclosed between 300 and 400 breaks. The calculated stress on the cable amounted to about 32,000 lb. per square inch, the designed working stress under full-load conditions being 80,000 lb. per square inch. It has accordingly been decided to replace the cables entirely, using the ordinary cold drawn material. . . . The news of these breakages led to an examination of the cables of the Detroit suspension bridge, . . . a number of the wires were found to be broken when examined. . . ." Tensile tests of some of the wire from the Mount Hope bridge in the Author's laboratory gave the following results: ultimate tensile strength 97·7 tons per square inch, reduction of area 43·3 per cent. A specimen was loaded at 92·8 tons per square inch for 26 days, and was then fractured; the tensile strength was 99·7 tons per square inch and the reduction of area 47·6 per cent. Tests under repeated torsional stresses gave a safe range of $\pm$ 20 tons per square inch, which is about equal to that of the best cold-drawn wire that the Author has been able to test, and corresponds to a fatigue-range of about $\pm$ 25 to $\pm$ 30 tons per square inch under bending stresses. The wires in the Detroit bridge had not been loaded to more than 5 tons per square inch, so that the cracks must have been produced, not by direct stress or repeated stresses due to the loading of the structure, but by some treatment received in manufacture or during erection.[2] *Fig. 16*, curves A and B, shows

[1] "Failure of Heat-Treated Wires in Suspension-Bridge Cables," *Engineering*, vol. 127 (1929, Part I), p. 461.

[2] It is well known that cold-drawn steels may become brittle after certain heat-treatments, but this does not appear to have been the explanation of the fractures in this case.

a very marked difference between the torsional properties of the heat-treated bridge wire and those of a cold-drawn wire (containing 0·65 per cent. of carbon, and drawn 75 per cent. reduction in seven passes) which might have been used. The cold-drawn wire also required about twice as many bends as the bridge wire to fracture it in the Arnold machine. Suggestions for suspension-bridges over the river Thames have been made, and in the future this type will be seriously considered for large-span bridges in Great Britain, especially where no interference with the waterway is desirable. The importance, therefore, of a thorough understanding of the properties of cold-drawn wire under static and repeated stresses is apparent.

Fig. 16.



NORMAL STRESS-STRAIN CURVES FOR TORSION; CURVE "A" FOR AMERICAN BRIDGE WIRE (HEAT-TREATED); CURVE "B" FOR COLD-DRAWN WIRE.

REPEATED-STRESS TESTS OF WELD-METAL AND WELDED JOINTS.

Welding is coming into use more and more in connection with built-up structures, and it is of the greatest importance that the behaviour of weld-metal and welded joints under repeated stresses should be investigated.

Weld-metal of the following composition :—

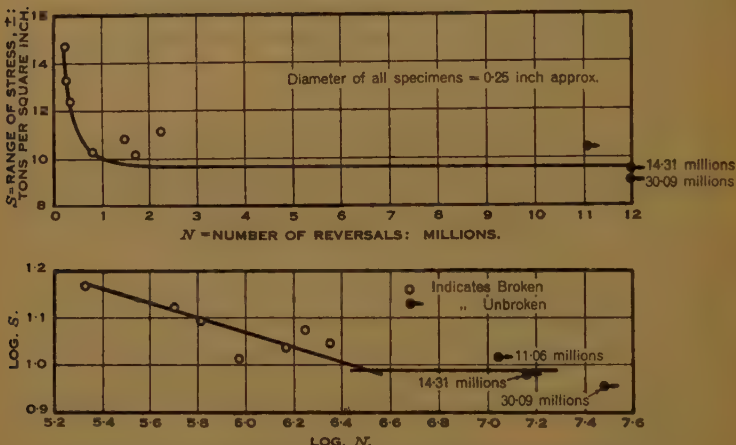
	C.	Mn.	Si.	S.	P.
Per cent. . . .	0·10	0·53	A trace	0·025	0·011

was laid down from electrodes on to a flat steel plate, forming ingots 12 inches long and about $1\frac{1}{4}$ inch by $1\frac{1}{2}$ inch in cross-section. Turned

test specimens were prepared from these ingots and tested in the "constant bending-moment" machine. The results of tests of ten specimens are plotted in *Figs. 17.*

The safe fatigue-range for 10 million reversals under bending is thus about ± 10 tons per square inch. The Author has tested other weld-metals and weld-metal specimens, in the Haigh machine, which gave values of only ± 5 to ± 7 tons per square inch for 10 million reversals.[1]

Figs. 17.



Fatigue-Tests (Bending) on All-Weld-Metal Specimens.

A tensile test of the weld-metal for which results are shown in *Figs. 17* gave the following results :—

Diameter of specimen : inch.	Temperature : degrees C.	Ultimate tensile strength : tons per square inch.	Elongation on 2 inches : per cent.	Reduction of area : per cent.
0·3	0	27·5	23·5	57
0·3	−55	30·4	33	68

Impact-tests in the Izod machine on standard round specimens 0·45 inch in diameter gave the following results :—

1st notch.	Izod value	85	foot-lbs.
2nd ,,	,,	78·5	,,
3rd ,,	,,	84·5	,,
	(Average)	82·7	,,

[1] Second Report of the Welding Research Committee. Proc. Inst.Mech.E., vol. 133 (1936), p. 5.

The fractures had a tough appearance, and the Izod values compare favourably with those obtained from mild-steel plates. Other weld-metals, tested as welds and not as all-weld specimens, have given impact values of only 23 foot-lbs.

The weld-metal used for the above-described tests was probably the most homogeneous and most free from blow-holes that the Author has examined, but the specimens were not uniform in quality and, as might have been expected, the fatigue-range at zero mean stress was much less than for a mild steel of the same tensile strength as the weld-metal. For such a mild steel the ratio of the safe fatigue-range for not less than 10 million reversals to the ultimate strength would be from 1 to 1·2. For the weld-metal that ratio was $\frac{20}{27{\cdot}5} = 0{\cdot}73$, which compares very favourably with the best fatigue-range obtained for the riveted or bolted joints. The Author has, however, obtained ratios of fatigue range for 10 million repetitions to ultimate tensile strength of as low as 0·5.

Figs. 18.

WELDED-PLATE TEST SPECIMEN.

The Author has designed and constructed a machine for testing welded plates in the black condition, and also a machine for testing girders. For the first of these machines, specimens of the form shown in *Figs. 18* are prepared. These are supported at A and B and the ends C and D loaded symmetrically so that the length AB is under a known constant bending moment and the top and bottom of the weld are alternately in tension or compression. The plates can be tested in the black state as welded, or after the machining. Tests on welds of the same plate with and without the excess weld-metal machined off have been made. In the machined condition a weld gave a fatigue-range of more than ± 8 tons per square inch for 10 million repetitions. Some of these specimens cracked in the weld and others in the plate. An extensive research has shown that under bending stresses the fatigue-range of welds is generally higher than in the Haigh machine. At zero minimum stress, for good welds, the fatigue-range varied from 9 tons per square inch to more than 15 tons per square inch. Results of some tests carried out in the

second machine have been incorporated in a Paper which has been accepted for publication by The Institution.

The Paper is accompanied by sixteen sheets of drawings and four photographs, from some of which the Figures in the text and the half-tone page-plate have been prepared.

Paper No. 5073.

"Road Traffic Considered as a Random Series."

By William Frederick Adams, B.Sc. (Eng.), Assoc. M. Inst. C.E.

(Ordered by the Council to be published with written discussion.)[1]

TABLE OF CONTENTS.

Introduction.

Road traffic may be considered in two different ways as similar to a distribution of points along a line, namely :—

(*a*) Each point representing the position of a vehicle at a given moment, and the line representing a length of road.

(*b*) The line representing a period of time and the points indicating the moments at which vehicles passed a given place.

The distribution is in both cases very irregular. Distributions showing similar irregularities are studied in the Theory of Probability under the names of "Random distribution" of points, or "Random series" of events, and it is a matter of considerable interest to examine how closely the actual distribution of road traffic agrees with the results of such theoretical studies.

Road traffic has already been assumed by Mr. John P. Kinzer[2] to form a random distribution, and by Mr. N. H. Martin [3] to be a random series, but in both cases these were in the nature of assumptions unsupported by proof. The Author considers that the assump-

[1] Correspondence on this Paper can be accepted until the 15th March, 1937, and will be published in the Institution Journal for October, 1937.—Sec. Inst. C.E.

[2] John P. Kinzer, "Application of Theory of Probability to Problems of Highway Traffic." *Abstract in* Proc. Inst. Traffic E., Fifth Annual Meeting (Oct. 2–3, 1934), p. 118.

[3] Messrs. Siemens' Bros. (Woolwich) Bulletin No. 3122, dated May, 1934.

tion of a random series leads to results of more practical utility, and has collected a large volume of evidence, showing that, under normal conditions, freely-flowing traffic corresponds very closely to a random series of events.[1]

A series of events is defined as "random" when :—

(a) each event, for example, the moment of arrival of a vehicle at a given point, is completely independent of any other event;

(b) equal intervals of time are equally likely to contain equal numbers of events.

Similar conditions, expressed in terms of space instead of time, define a random distribution.

General Considerations.

In the Theory of Probability it is shown that for a random series the probability of any given number x of events occurring in a given time is given by

$$P(x) = e^{-m}.m^x/x! \quad \text{(Poisson's Law.)} \quad . \quad . \quad . \quad (1)$$

where $P(x)$ denotes the probability of x events occurring,

m	,,	mean number of events expected in the given time.
e	,,	base of Napierian logarithms $= 2{\cdot}71828 \ldots$
$x!$	,,	product of the first x natural numbers $= x(x-1)(x-2) \ldots 3.2.1$

From this it follows that, if road traffic were a random series and if n equal periods of time were considered, the probable number of these periods in which x vehicles arrive

$$= n.e.^{-m}.m^x/x! = n.P(x) \quad . \quad . \quad . \quad . \quad . \quad (2)$$

The Author has verified by observations of actual traffic ranging in volume from 70 to 1,400 vehicles per hour that freely-flowing traffic normally conforms to the above law. *Fig. 1*, for example, shows a comparison between the observed numbers of 10-second periods in which given numbers of vehicles arrived and those calculated from the above formula for two typical traffic streams of 222 and 618 vehicles per hour respectively. The agreement in these and in many other similar cases is sufficiently good to justify the working assumption that road traffic is normally a random series.

It may also be shown that the lengths of intervals of time elapsing

[1] Examples of this are given in the MS. of the Paper, which may be seen in the Institution Library.—Sec. Inst. C.E.

between the arrivals of consecutive vehicles should be distributed in accordance with the following law, which may be termed the "Exponential Law of Intervals."

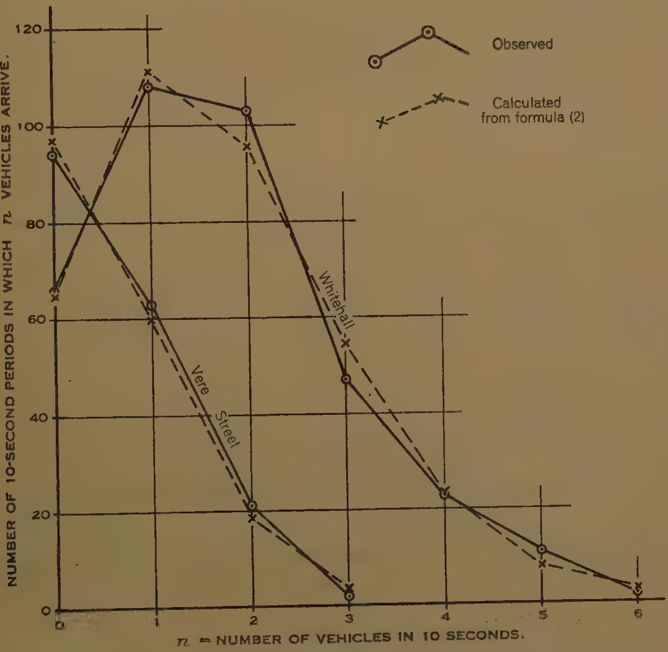
The number of intervals greater than t seconds $= a.e^{-Nt}$ . . (3)

where a denotes the total number of intervals considered,

N ,, average number of vehicles per second (generally fractional),

and e, as before, has its normal mathematical meaning.

Fig. 1.



DISTRIBUTION OF NUMBERS OF VEHICLES ARRIVING DURING PERIODS OF 10 SECONDS.

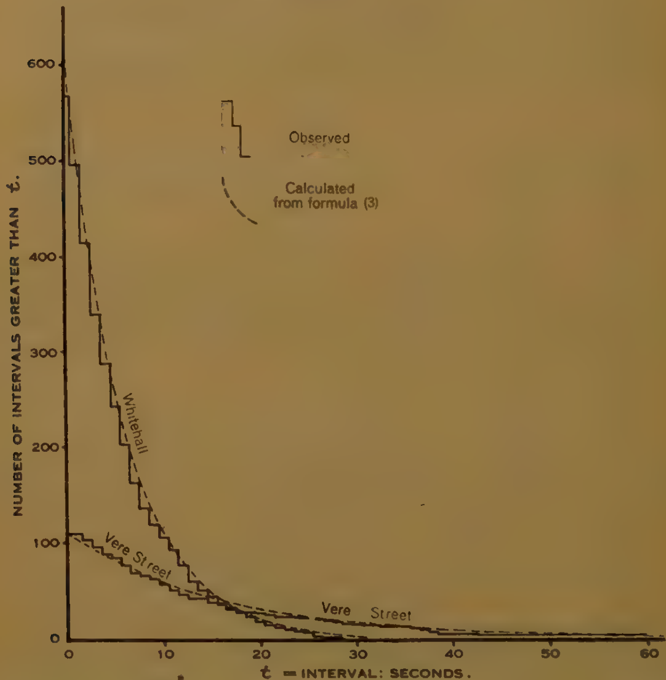
Fig. 2 (p. 124) compares the observed and calculated distributions of intervals for the two cases already considered, and it will again be seen that the agreement gives good support to the working assumption.

These formulas may be themselves employed for the solution of practical problems.[1] For practical purposes, however, the formulas

[1] Completely worked examples are given in the MS. of the Paper, which may be seen in the Institution Library.—SEC. INST. C.E.

are rather cumbrous, and Tables[1] may be used, or the values required may with reasonable accuracy be read from Fig. 3, Plate 1 (which has been prepared from Tables computed by Mr. D. A. de C. Bellamy), according to the methods outlined below.

Fig. 2.



Distribution of Intervals Elapsing Between Arrivals of Successive Vehicles.

Method of Using Curves.

The curves in Fig. 3, Plate 1 may be used as follows to find :—

(1) *Probability that* x *or more vehicles will arrive in a given time.*—Through the value of m on the base line a vertical line should be drawn to intersect the curve for the given value of x. From the point of intersection a horizontal line must be drawn, and the required probability may be read off where this line cuts the left-hand scale.

[1] K. Pearson, "Tables for Statisticians and Biometricians," Tables LI and LII. Cambridge University Press, 1930.

T. C. Fry, "Probability and its Engineering Uses," Appendices VI and VII, pp. 458 and 463. London, 1928.

(2) *Probability that less than* x *vehicles will arrive in a given time.*—This procedure is as just described, but the results should be read on the right-hand scale.

(3) *Probability that just* x *vehicles will arrive in a given time.*—The probability that x or more vehicles will arrive must be found as described in (1), and from this must be deducted the probability, similarly found, that $(x + 1)$ or more vehicles will arrive. The difference is the required probability.

(4) *Probability that the number of vehicles in a given time will lie between given limits.*—If it is required to find the probability that the number of vehicles will be y or more and will not exceed z, then, as above, the probability that the number of vehicles is y or more must be found, and also the probability of its being $(z + 1)$ or more. The difference is the required probability.

(5) *Probability that any interval between vehicles will be greater than* t *seconds.*—The value of m, the average number of vehicle arrivals in t seconds, must be found, and from the curve for x=1 the probability required may be read on the right-hand scale.

(6) *Probability that no vehicle will arrive in given time.*—This is a special case of (2). "No vehicle" means "less than one vehicle," so the value required is read from the curve for $x = 1$ on the right-hand scale.

(7) *Probability that any vehicles will arrive in given time.*—This is a special case of (1) above. The arrival of any vehicles is the same as that of one or more vehicles, so that the required value is found from the curve for $x = 1$ on the left-hand scale.

It will be seen that cases (5) and (6) are in effect the same, although differently expressed.

Examples.

The following examples of practical applications are discussed in order to provide illustrations of the methods of working employed :—

Example 1.—A simple cross-road junction is to be controlled by traffic signals of fixed-cycle type. There is one heavy right-hand turning movement amounting to 165 vehicles per hour. The layout of the junction is such that if three or more vehicles making the right-hand turn arrive during one cycle they obstruct other vehicles while waiting to make the turn. If the control cycle is 45 seconds, how frequently will obstruction result from this cause ? If a special provision were introduced in each cycle to assist this right-hand turn, how frequently would it be introduced unnecessarily ?

The mean number of vehicles making the right-hand turn per cycle $= m = 165 \times 45/3600 = 2{\cdot}06$.

By method (1) above, the probability of three or more vehicles arriving in one cycle is found from Fig. 3, Plate 1, to be 0·340. Obstruction will therefore result in 34 per cent. of the cycles, *i.e.* 34 per cent. of 3,600/45 times per hour

$$= \underline{27{\cdot}2 \text{ times per hour.}}$$

This is sufficiently frequent for it to be obvious that some special means of controlling this turn is required.

If such means is introduced, it will be unnecessary only when during one complete cycle no vehicle making this right-hand turn happens to arrive. From Fig. 3, Plate 1, using method (6) above, it is found that for $m = 2{\cdot}06$, $P(0) = 0{\cdot}128$; that is, in 12·8 per cent. of the cycles the provision will be unnecessary; or, it will be unnecessary $0{\cdot}128 \times 3600/45$

$$= \underline{10{\cdot}2 \text{ times per hour.}}$$

By using Poisson Tables the above probabilities are found more accurately to be $P(3 \text{ or more}) = 0{\cdot}3396$, and $P(0) = 0{\cdot}1276$, respectively.

Example 2 concerns a cross-road junction controlled by fixed-time traffic signals, and the following particulars of timings and traffic volumes are given :—

Road N carries 243 vehicles per hour; Road S carries 220 vehicles per hour	Green period for these roads	23 seconds.
	Amber	2 ,,
Road E carries 75 vehicles per hour; Road W carries 173 vehicles per hour	Green period for these roads	17 ,,
	Amber	2 ,,
	Total cycle . .	44 ,,

It is required to find, for each traffic phase, what proportions of signal periods will be unnecessary, too long, too short, and correct.

It is found by simple applications of Poisson's law and with the aid of Tables that the proportion of green periods that are

unnecessary is	0·4 per cent. for N and S and	4·8 per cent. for E and W
too long ,,	97·9 ,, ,, ,,	88·6 ,, ,, ,,
too short ,,	0·5 ,, ,, ,,	2·1 ,, ,, ,,
correct ,,	1·2 ,, ,, ,,	4·4 ,, ,, ,,

Full details of the working are given in the MS. of the Paper. The timings represent fair normal practice for the traffic volumes given.

Example 3.—It is required to find the average delay experienced by pedestrians attempting to cross a normally-distributed traffic stream of N vehicles per second on the assumption that each pedestrian has to wait until the occurrence of an interval between vehicles exceeding t seconds (t is not necessarily the time taken to cross the road, and is usually substantially less).

In the course of solution of this problem the following general results are shown to be true for traffic distributed normally (that is, as a random series) :—

(I) Proportion of time occupied by intervals greater than t seconds $= e^{-Nt}(Nt + 1)$.

(II) Proportion of time occupied by intervals less than t seconds $= 1 - e^{-Nt}(Nt + 1)$.

(III) Average length of all intervals greater than t seconds $= (1/N + t)$ seconds.

(IV) Average length of all intervals less than t seconds $= 1/N - t.e^{-Nt}/(1 - e^{-Nt})$ seconds.

These formulas, and others used in the course of the Paper, are expressed in terms of e^{-Nt} rather than of e^{Nt} for the sake of greater convenience in using Poisson Tables, as explained in the MS. of the Paper.

From the above general results it is shown that :—

the proportion of pedestrians delayed is $(1 - e^{-Nt})$,
the average delay these suffer is $1/N.e^{-Nt} - t/(1 - e^{-Nt})$ seconds,
the average delay suffered by all pedestrians, including those who find themselves able to cross without waiting, is $1/N.e^{-Nt} - 1/N - t$ seconds.

Observations on Results of Example 3.

These formulas are rather remote deductions from the original assumptions, and it is desirable that they should be checked by comparison with delays found in actual practice. This, however, presents difficulties, because as the value of t (minimum gap in traffic permitting safe crossing) is not known the results obtained by observation cannot be compared directly with those given by calculation. The following indirect method provides both a test of the consistency of the formulas with actual fact and a means of finding t.

At a well-defined pedestrian crossing, let observations be taken of the following :—

Traffic volume over the crossing, N vehicles per second.
Total number of pedestrians observed to use the crossing, P.
Number of pedestrians able to cross without delay, p.
Total delay sustained by all pedestrians, D seconds.

Particular care must be taken to include under p no pedestrians who are in the slightest degree delayed, as this would falsify the value of t found by method (a) below.

The following possible methods of determining the value of t are then available :—

(a) From the proportion of pedestrians not delayed, by solving equation $p/P = e^{-Nt}$.

(b) From the average delay to all pedestrians, $D/P = 1/Ne^{-Nt} - 1/N - t$.

(c) From the average delay to delayed pedestrians, $D/(P - p) = 1/Ne^{-Nt} - t/(1 - e^{-Nt})$.

(d) Roughly, by direct inspection, noting the lengths of gaps in traffic and whether pedestrians do or do not cross in each gap. Excluding all values which do not occur with some frequency, the value of t lies between the shortest remaining gap in which pedestrians do cross and the longest remaining gap in which they do not. This method does not give rigid limits, but serves to show the range in which the value of t should lie.

If the four values of t found as above agree reasonably well, the pedestrian-delay formulas used in finding t are consistent with the facts, and a good value for t can be found by averaging the results given by methods (a), (b), and (c) above.

Observations have been made at a number of sites in the central London area, with the results summarized below :—

TABLE I.

Site.	Traffic: vehicles per hour.	Average time to cross : seconds.	Approximate width of crossing: feet.	Observed values of		
				P	p	D : seconds
Sloane Street .	360	3·92	17	228	155	180
The Mall . .	1132	6·15	30	61	16	323
Kingsway . .	529	5·20	25	316	158	680
Whitehall . .	964	7·40	37	76	30	214
Birdcage Walk .	554	7·62	36	73	40	108

Table II gives the results worked out from these figures. The agreement of the last four columns is reasonably good. The value of t appears to depend on the road width, the traffic volume, and possibly on other factors. It may be accepted that the formulas are confirmed, but further research is necessary into the value of t appropriate for given conditions.

TABLE II.

Site.	Proportion of pedestrians not delayed: per cent.	Average delay to		Value of t by method			
		All pedestrians: seconds.	Delayed pedestrians: seconds.	(*a*): seconds.	(*b*): seconds.	(*c*): seconds.	(*d*) (rough only): seconds.
Sloane Street .	68·0	0·79	2·47	3·76	3·73	3·71	3·8–4·0
The Mall .	26·2	5·30	7·18	4·26	4·48	4·51	4·2–4·8
Kingsway .	50·0	2·15	4·30	4·72	4·78	4·81	4·2–4·8
Whitehall .	39·4	2·82	4·66	3·48	3·74	3·93	3·6–4·2
Birdcage Walk	54·8	1·48	3·27	3·93	3·94	3·96	3·2–4·0

CONCLUSION.

The MS. of the Paper concludes with some discussion of conditions under which the traffic distribution differs from that of a random series. Freely-flowing traffic is found to conform so well to the distribution given by a random series that the latter may be described as "normal." Departures from normal distribution are produced by disturbance of the free and uniform flow of traffic by one or more of the following causes :—

(1) Sudden increase or decrease in the traffic using the road.
(2) Control by police or traffic signals at an adjacent site.
(3) Difficulty in passing other vehicles freely, owing to narrow roads, tramways, sharp bends, etc.
(4) Saturation, which, however, is in practice a rare condition.

These are the only disturbing influences shown by the traffic observations so far available. Further effects of a similar kind may be discovered when more detailed studies are made.

The effects produced by conditions (1), (2), and (3) are somewhat similar in type, since their influence is that of producing alternate periods of greater and smaller traffic flow. Numbers of vehicle arrivals considerably greater or less than the average occur more frequently than with normal traffic, while numbers near the average occur relatively less frequently. The longer and shorter intervals are also more frequent, and intervals of nearly average length less frequent, than with normal traffic.

Saturation of traffic flow produces very different effects. It has been a matter of some difficulty to obtain observations of this influence, since it does not become pronounced until a traffic of over 1000 vehicles per hour per traffic lane is reached, and this represents a rate of flow seldom attained. Traffic flow in streets is very rarely saturated, since the amount of traffic which a given street can carry is usually limited rather by bad design of junctions

or the effect of control than by the width of the street itself. However, such figures as have been obtained under nearly saturated conditions show that numbers near the average are more frequent and those remote from the mean are less frequent than with normal traffic. There is a reduced number of longer intervals, and short intervals are considerably more frequent than under normal conditions.

The Paper is only an introduction to the subject, since there is room for much research into the results of both random and non-random traffic distributions. A number of results are already in use among engineers concerned with traffic control.

In summary, the Author considers that

(*a*) Road traffic normally conforms to the laws true of random series, and may therefore be assumed to form a random series.

(*b*) The laws of random series may be applied to road-traffic questions in order to obtain numerical answers to specific traffic problems, or to decide questions of general principle.

(*c*) Applications of these laws are already in use, and further applications will no doubt be made.

(*d*) The results calculated from these laws are true of normal traffic, but are only a rough approximation for traffic conditions disturbed by variable traffic flow, control, difficulty of passing, or saturation. The general effects of such disturbing influences are known, and further research may result in more accurate formulas being found even for such disturbed conditions.

The MS. of the Paper includes Appendixes [1] giving two sets of traffic observations in full, with details of the methods of working and checking. Explanations of the various methods and results of the Theory of Probability used are given in footnotes.

The Author acknowledges assistance given by Mr. N. H. Martin, and by his colleagues of the Ministry of Transport, but disclaims any official support for the views expressed. The theory of telephone traffic has provided many suggestions, but obligations in this direction are too numerous to be concisely expressed.

The Paper is accompanied by four sheets of drawings, from some of which Plate 1 and the Figures in the text have been prepared, and by two Appendixes.

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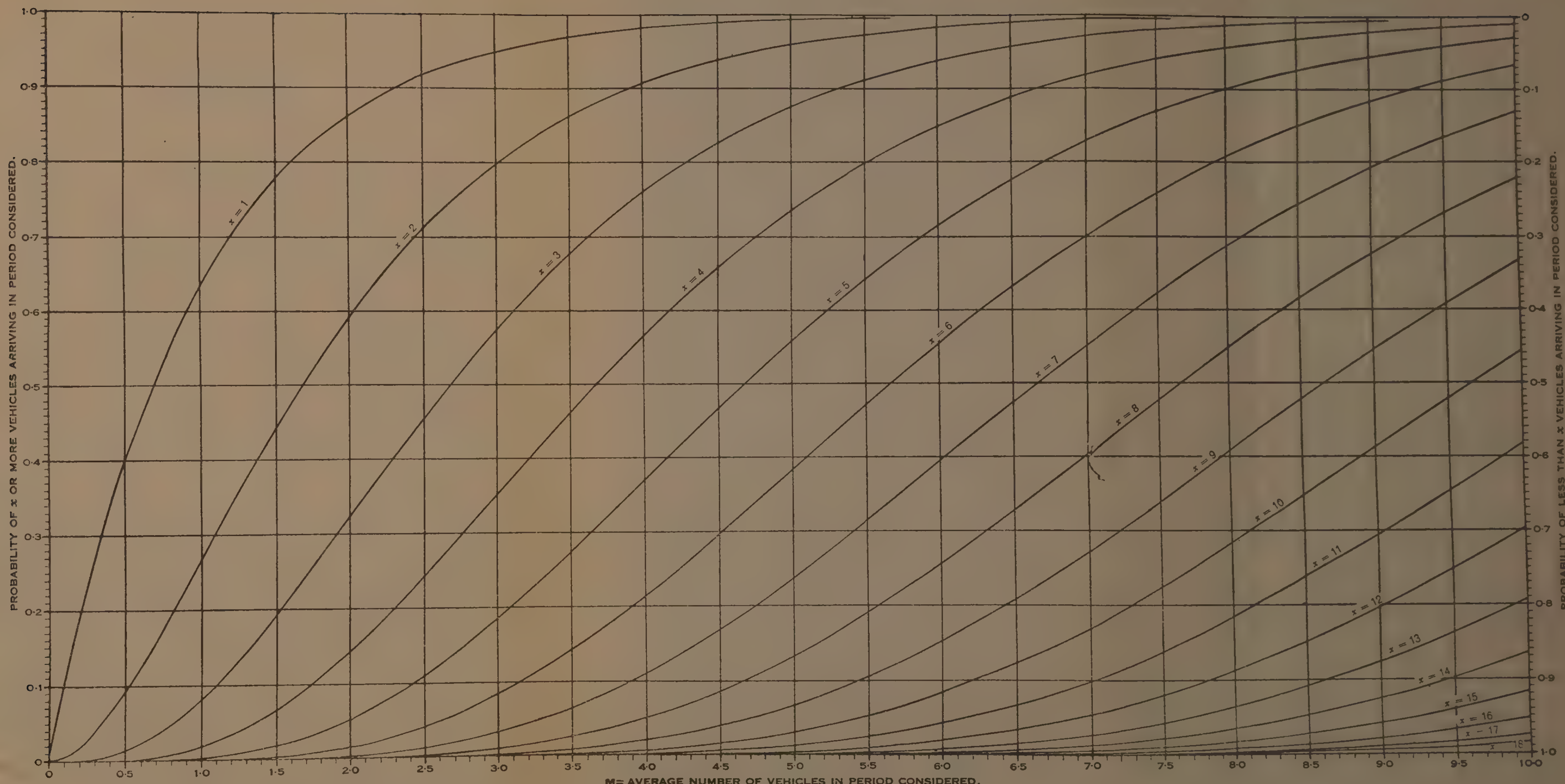
[1] These may be seen in the Institution Library.—Sec. Inst. C.E.

ROAD TRAFFIC CONSIDERED AS A RANDOM SERIES.

FIG. 3.



Note: Normal (Random) traffic distribution is assumed.

DIAGRAM SHOWING PROBABILITY THAT THE NUMBER OF VEHICLES ARRIVING IN A GIVEN TIME WILL BE *X* OR MORE (OR LESS THAN *X*).

The Institution of Civil Engineers. Journal. November, 1936.

WATERLOW & SONS LIMITED, LATE THOS. KELL & SON, LONDON.

W. F. ADAMS.

Paper No. 5039.

" The Active and Passive Pressures of Sea-Sand Behind a Vertical Wall."

By Arthur Allison Fordham, Ph.D., B.Sc., Assoc. M. Inst. C.E.

(Ordered by the Council to be published in abstract form.)[1]

In the design of retaining walls it is usual to assume that the lateral pressure of the earth backing increases uniformly with the depth below the surface in the same way as water-pressure. The experimental research described in this Paper was carried out in order to find the actual distribution of pressure behind a vertical wall, and the general law of variation of pressure with depth of sand.

A vertical steel-plate cantilever 2 feet 6 inches high, 7 feet 6 inches long and $\frac{1}{4}$ inch thick was used to form one side of a bin which was loaded with sand. Measurements of the horizontal deflection of the cantilever by means of dial gauges gave the form of the deflection-curve, from which the distribution and magnitude of the horizontal pressure were calculated.

The distribution of active horizontal pressure was investigated by three different methods, all of which agreed in indicating that the pressure-intensity at any depth d below the surface of the sand can be expressed as pd^n, where n is less than unity. This distribution gives the centre of pressure higher than one-third of the depth. The three methods of investigation referred to are described below :—

(1) The deflection curve taken up by the cantilever-plate test-wall was obtained for a large number of tests. The mean curve through the experimental points was found to agree closely with that calculated for $n = 0{\cdot}8$.

(2) The actual deflections of the test-wall at the surface-level of the sand were measured and plotted for various depths of sand-backing.

(3) The deflection-curve due to a uniformly-distributed superimposed load of bricks upon the sand-backing was measured and showed clearly that the effect of the superimposed load was not a uniformly-distributed pressure behind the wall, as would be the case if $n = 1$.

[1] The MS. and drawings can be seen in the Institution Library.

All three methods agreed in suggesting that for the sand used in the tests the active horizontal pressure-distribution may be represented by the following law:

Intensity at depth d below the surface $= pd^{0 \cdot 8}$,
Height of centre of pressure $= 0 \cdot 357\ H$,

where H denotes the height of the wall and p the horizontal pressure-intensity at unit depth calculated by the wedge theory or Rankine's formula.

The application of this law of variation of active pressure with depth is considered in the case of actual walls, and it is shown to be closely correct for a depth of 48 feet below the surface in the case of a tunnel heading on the Metropolitan railway at Campden Hill, particulars of which were given by Sir Benjamin Baker in 1881.[1]

The tests for passive pressure were carried out by bending the cantilever plate back against the pressure of the sand, a horizontal force being applied at the top. The following results were obtained for passive pressures:

(1) The distribution of horizontal passive pressure was practically uniform.

(2) The magnitude of the passive pressure actually developed depended upon the movement of the head of the wall at sand surface-level.

(3) The maximum ratio of the total horizontal passive pressure to the total horizontal active pressure for any depth of sand was about 3 to 1.

It is to be noted that the passive-pressure test results apply only to a flexible vertical cantilever-wall, fixed at the base, and the pressure developed is much less than would arise in the case of a wall forced bodily against the resistance of the sand. In all cases of passive pressures a very considerable wall-movement is required to develop the maximum resistance of the filling.

The tests were carried out by the Author in the Engineering Department of the University College of Swansea.

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[1] Minutes of Proceedings Inst. C.E., vol. lxv (1880–81, Part III), p. 158.

Paper No. 5042.

"Regirdering of Bridges on the Great Indian Peninsula Railway."

By William Hood, Assoc. M. Inst. C.E.

(Ordered by the Council to be published in abstract form.)[1]

The Paper gives a brief history of circumstances leading up to the regirdering of the major bridges on the north-east main line of the G.I.P. Railway, and indicates the development of the methods finally adopted for carrying out the work under traffic on nine single-line bridges of spans ranging from 60-foot plate-girder deck and through spans to 200-foot through-spans of wrought-iron multiple-latticed girders on the highest railway bridge in the plains of India.

The substructures in all cases were known to be sound and capable of carrying superstructures to the requisite heavy standard of loading to which the superstructures were to be designed, and therefore on economical grounds it was necessary to develop schemes to carry out the regirdering under traffic on the existing substructures.

The straightforward plan of staging each span from the river-bed level had the great disadvantages of expense and the limitation of the work to the dry season of about 5 months' duration, and it was obvious that some form of service span, easily launched from span to span, was preferable and cheaper. Fortunately, the piers of all the bridges were built with cut-waters up-stream and down-stream, and therefore enveloping service-spans with the bearings on the cut-waters could appropriately be used.

In the preliminary designs for the service spans two different methods of launching them were devised, as some of the bridges were of pony-truss-type spans whilst the others were of the conventional through-type with overhead bracing. As the work proceeded, however, it was found possible to dispense with the method proposed for the latter type, which was definitely slower and more costly.

The method of building the service span on the embankments at the entrances to the bridges, the launching of it on two steel towers mounted on ordinary 4-wheeled timber-trucks of 16 tons capacity, and clearing the line for traffic within an hour, are described

[1] The MS. and drawings can be seen in the Institution Library.

in detail. Then follows a description of the change from carrying traffic on the existing span to carrying it on the service-span floor, and of the demolition of the existing floor.

The next operation of removing the existing girders complete and launching the new girders, assembled and riveted complete in the girder-yard, into position on the bridge without the use of large cranes, is a feature of the scheme, and this was largely responsible not only for the speed of the regirdering but also for the low cost per ton of new steelwork erected under traffic.

The normal slow and costly process of breaking up the old girders and assembling and riveting the new girders in position on the service-span in the short intervals between trains was avoided by the design of two special trollies running on the 5-foot 6-inch gauge track to carry the girders between the bridge and the girder-yard. On the bridge the existing girders were lifted to the requisite height by means of four 20-ton hand-operated pulley blocks, two at each end, mounted on trollies running on transverse gantries on the top of the service span, and, after being traversed to the centre of the track, they were lowered on to the special trollies which were run in underneath the girder from each end. The complete girder was then towed to the girder-yard, removed from the trollies by similar gear mounted on gallows in the yard, and then broken up.

The new main girders, completely riveted in the girder-yard, were placed in position in the bridge by the same gear and trollies, and the reverse operation of changing from service span-floor to new span-floor carried out. Each of the operations indicated was completed within the block period, which averaged about 2 hours between trains. At the Chambal bridge the rate of regirdering by this method was reduced, after some experience was gained, to the short time of 8 working days for the 200-foot spans.

The modifications necessary for bridges of different types are described, together with notes on difficulties which arose in the actual work of regirdering each of the types. The necessary site arrangements, including brief descriptions of girder-yards, operating platforms, camps, and plant, are also dealt with.

Short accounts of the methods by which deck and through plate-girder 60-foot spans were regirdered in the course of the regirdering of the major spans by the service span are also given, together with a description of the regirdering of 80-foot through truss-spans by trestles erected in the dry river-beds, traffic being carried by the existing and new floor systems which were cradled on 30-foot plate-girder spans supported by the trestles.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

There appears below an interesting report giving the present position of the science of Soil Mechanics and Foundation Engineering by Mr. L. F. Cooling, with whose work on soil mechanics at the Building Research Station the Institution Sub-Committee on Earth-Pressures is associated. Attention is drawn to the plea for facilities for observing the settlement of actual buildings during and immediately after construction. An invitation to co-operate with the Building Research Station in this respect has already appeared in the Journal, and it is hoped that the value of such co-operation, as evidenced in this report, will make a strong appeal to all concerned. The report also contains a reference to the effect of pile-driving on the soil, a complementary aspect of the problem to that which is being considered by the Joint Sub-Committee on Pile-Driving.

Sub-Committee on Earth-Pressures.

Report on the First International Conference on Soil Mechanics and Foundation Engineering.

By L. F. Cooling, M.Sc.

The First International Conference on Soil Mechanics and Foundation Engineering, which the Author was privileged to attend on behalf of the Building Research Station and the Road Research Laboratory, was held at Harvard University, Cambridge, Massachusetts, U.S.A., from the 22nd to the 26th June, 1936. The primary object of the Conference was to collect and co-ordinate the mass of information which has been obtained by soil workers all over the world, and to prepare the way for a closer co-operation, not only between workers in different countries, but also between the practising engineer and the research worker. The need for this co-operation between engineer and research worker is emphasized by the fact that in the present position not only is it difficult for the engineer to keep in touch with the latest developments in soil science, but in addition progress in research is handicapped because many of the problems in soil mechanics require for their solution information which cannot be obtained in the laboratory.

The attendance at the Conference must have been most gratifying to its organizers, for over two hundred members from some twenty different countries participated, research workers and practising engineers being nearly equally represented. The scope of the problems considered covered a wide field in engineering practice, and such a large number of Papers were submitted that it was not possible to present them in session, but instead abstracts were printed in advance in two large volumes. A third volume, containing discussions and some complete Papers, will be published towards the end of the year.

The outstanding impression formed as the result of attendance at the meetings was the extent to which soil science has already established for itself a place in engineering practice in other countries, notably in the United States. In many large engineering projects and road-construction schemes soil studies are carried out in a systematic manner as part of the normal routine. It is also significant of the status of the science that in many of the American universities a course in soil mechanics is included in the curriculum of engineering schools. While it is evident that the research workers at the Conference were only too conscious of the amount of research still called for, yet it was equally evident that the engineers felt that already soil science can offer them much information of important practical value.

An outstanding result of the Conference was that it brought about a realization of the fact that, in spite of the wide field of engineering practice which was covered, the study of soil mechanics forms a common fundamental basis on which the many different problems can be most usefully attacked. In addition, the enthusiasm displayed by the members, laboratory workers and engineers alike, showed that the discussions and personal contacts had brought a realization of unity of effort which was most stimulating. Tribute must also be paid to the work of Mr. A. Casagrande and his colleagues to whom, as organizers, so much of the success of the Conference was due, and to the admirable arrangements for the reception and comfort of members made by the authorities of Harvard University.

The published volumes of the Conference will represent a rich mine of information on the subject generally, and will merit very close study by those interested. These Proceedings were divided into fifteen Sections, and while comments are made in the following notes on each Section, references of a lengthier nature are made to the contributions bearing on the settlement of buildings and highway construction, since on the one hand these are two of the most important Sections of the Conference from the practical point of view

and on the other hand they are Sections with which the Author himself was particularly concerned.

A fuller account of some of the other Sections was published recently.[1]

Section A. Reports from Soil-Mechanics Laboratories.

The Reports submitted to the Conference under this heading are in the nature of replies to a questionnaire concerning the organization, equipment, experimental methods, and lines of research being pursued by soil laboratories. They reveal that soil research is being actively pursued in some twenty fully-equipped laboratories in England, America, Germany, France, Holland, Austria, Egypt and Japan. With regard to the experimental methods which are being adopted, there seems to be a fair degree of uniformity, and, of course, universal agreement cannot be expected. Much of the work carried out is in the nature of fundamental research, but quite a large proportion is done in a consulting capacity.

Section B. Exploration of Soil Conditions and Sampling-Operations.

The Papers in this Section deal mainly with the description of apparatus used for obtaining "undisturbed" samples of cohesive soils. In most cases the methods used are similar in principle and utilize a sampler, which consists of a cylindrical tube with a cutting edge which is either forced into the soil by steady pressure or hammered in. A few novel methods are described but these are open to serious objections. There is, however, still need for further improvements to ensure that really "undisturbed" samples are obtained in all cases.

Section C. Regional Soil Surveys for Engineering Purposes.

These Papers are of interest mainly because they demonstrate the utility of a preliminary soil survey of a large area before commencing important civil engineering work. An interesting example is that of Flushing Meadow, New York, which was proposed as a site for a public park and may be used as the site for the 1939 World's Fair. This meadow-land is low and marshy and it was necessary to place several million cubic yards of fill to make it suitable. The exact amount of fill required could not be determined until the extent of the consolidation under the weight of the fill could be estimated. Consequently the whole of this area was surveyed, borings taken and soil tests performed and, based on the results

1 "Soil Mechanics and Foundation Engineering." *The Engineer*, vol. 162 (1936), pp. 175 and 199.

obtained, recommendations were made for the development of the site. Other interesting Papers described tests performed on a number of samples to determine the properties of soils in the regions around Cairo and Chicago.

Section D. Soil Properties.

The Papers in this Section are mainly concerned with the laboratory investigations of those properties of soils which are not yet fully understood. Most of them deal with the properties of cohesive soils and a number of these are concerned with methods of investigating the shear-strength, the study of which presents many difficulties. Of the other important soil properties, methods for the investigation of consolidation, permeability and compression are fairly well standardized and are based on the methods developed by Professor K. von Terzaghi. Some interesting points are, however, raised in connection with the effects of the speed of loading, duration of loading and temperature. A rather different aspect of research on soil properties is put forward in a Paper dealing with the influence of the chemical nature of clays on their mechanical behaviour.

Section E. Stress-Distribution in Soils.

Papers in this Section are very largely of a mathematical nature and describe some analytical methods for determining stress-distribution based on the theory of Boussinesq. The main practical interest of this Section concerns its application to the forecasting of the settlement of buildings, a subject considered in the next Section. One Paper of a practical nature records measurements of soil pressure on a tunnel-lining.

Section F. Settlement of Structures.

Before considering the Proceedings of the Conference in relation to building-foundations it is desirable, first of all, to set down generally the position of research in regard to available methods of obtaining the soil data necessary to meet the demands of the foundation engineer, then to point out some of the many problems which still have to be solved, and finally to refer to a few interesting examples of the useful application of knowledge derived from soil research to the solution of foundation problems.

In the past engineers had hoped that it would be possible to evolve certain empirical laws by means of which foundation problems could be solved with an exactness approaching that obtainable with problems arising in connection with simpler materials such as steel or concrete. The research that was carried out was directed towards

this end, but it is now realized that the hope was unwarranted, since the factors involved are of such a very complex nature. The aim at the moment is rather to understand the actions which take place when a building settles. This necessitates the exploration of all the conditions which obtain at a particular site, taking into account the soil strata to an appreciable depth below the level of the footings, for foundation soils are very rarely uniform. The main fundamentals of soil action are fairly well understood, and tests have been developed, and are now nearly standardized, by means of which measurements can be made on soil samples which enable at least a qualitative estimate of the settlement in a particular case to be formulated. A complete understanding, however, of all the factors involved has not yet been obtained, so that the research is at present in a development stage, and at a stage at which it is essential to obtain data of actual observations on buildings which will enable comparisons with predicted settlements to be made.

The Proceedings of the Conference contain many interesting accounts of investigations of this kind carried out abroad. One Paper contains a report of work which has been undertaken in Cairo, Egypt, in which fourteen buildings are under observation. Some of these are on raft-foundations and others are on piles, and the local soil consists of clay overlying sand at approximately 10 metres depth. Comparisons made between observed and calculated settlements have shown that in some cases there is satisfactory agreement both as regards rate and distribution of settlement. In other cases the theoretical settlements appear to be slightly too high. Heavy buildings on the clay have shown settlements of between 8 and 15 centimetres, which in some cases have resulted in damage not only to the buildings themselves but also to adjoining structures. Buildings on piles penetrating well into the sand seem to have shown very small settlements.

A report from Freiberg, in Germany, records that twenty buildings are under observation. Comparisons between the observed and calculated settlements indicate that the agreement depends largely on the type of soil strata encountered. With stiff clays or soft plastic clays, especially when present in thin layers sandwiched between sand layers, the observed settlements check with the predicted values ; for loose silt, loam and similar soils which are loosened up when removed from the ground, the actual settlement was found to be appreciably lower than the computed values ; for very soft clays, peats, etc., if present in thick layers, the observed settlements were found to be larger than the theoretical values, probably because of lateral flow.

A report from Vienna, Austria, records that twenty structures in

the vicinity of Vienna are under observation. In addition, Professor von Terzaghi, who is in charge of the laboratory, receives settlement records of many other buildings. In some cases the agreement between observed and predicted settlement is satisfactory, but important differences are noted in other cases.

A report from France states that two buildings have been under observation, and that satisfactory agreement was obtained between observed and predicted values. The soils laboratory at Harvard receives settlement-records of a number of structures. The experience in this laboratory has been that for inorganic clays the predicted settlements are in good agreement with observations. In many cases, and particularly with large loaded areas, settlement due to lateral displacement in clay can be entirely neglected. With some types of silt and silt clays no amount of care in sampling can prevent serious disturbance of the structure of the material, and hence tests of these materials give values for the compressibility of the soil which would be higher than those the soil would possess in its natural state in the ground. A Report from Professor W. S. Housel, of the University of Michigan, states that ten major structures are under observation. Professor Housel approaches the subject of settlement in a manner rather different from most workers in this subject, basing his results on a system of bearing-tests. He claims agreement between observed settlement and values calculated by his methods.

From this brief account it can be seen that there is considerable activity in this field in other parts of the world. The fact that discrepancies are found between observed and calculated values of settlement in certain cases is not wholly unexpected, in view of the wide range of materials represented by the term "soil," and also the complicated nature of the factors involved. While, however, it is necessary, in order to solve these difficulties, to inquire more fully into the validity of the assumptions made in calculating theoretical settlements and into the interpretation of test-results, the most urgent need appears to be a more widespread pursuit of investigations of the type already described. With this object in view the Conference sent out an appeal to practising engineers for active co-operation in collecting the necessary data. It will be recalled that a similar appeal was issued recently by the Institution Research Committee.[1]

Nevertheless the progress which has already been made in this field is important, and it is significant that a number of examples

[1] Joint Sub-Committee on Pile-Driving. Journal Inst. C.E., vol. 2 (1935–36), p. 593. (April, 1936.)

were quoted by engineers in which the application of the results of soil tests had assisted materially in solving difficult foundation problems. These examples are described in later Sections of the Proceedings (Sections N and Z).

In the course of the discussions it was pointed out that load tests on single piles or footings of small area may give no idea of the eventual settlement of a building. Examples were cited in which appreciable settlements had been observed in the building, even though the pile loads or unit pressures had been limited to a value at which during load tests negligible settlement had occurred. This is due to the fact that with a large loaded area or groups of piles the pressure affects soil strata at a considerable depth below the level of the footings or the base of the piles, whereas with a small area or single pile the influence is localized. Thus with a large loaded area the influence of the pressure may extend to a deep-seated, very compressible, layer of soil which would cause appreciable settlement. Small-area tests would give no evidence of the presence of this strata and the results would therefore be misleading. It is significant that in many cases of serious settlements the cause has been traced to a deep-seated compressible layer.

In concluding the report of this Section it may be noted that the Proceedings of the Conference made it clear that in no field of practical work are the possibilities of soil science more promising than in that of building and bridge foundations.

Section G. Stability of Earth and Foundation Works and of Natural Slopes.

Many of the Papers in this group are concerned with the theoretical analysis of the stability of natural slopes. They deal with the calculation of the safety against sliding along a surface of an assumed shape, in some cases a circular sliding surface after the method of K. Petterson and in others a surface with the shape of part of a logarithmic spiral after the method of L. Rendulic. The influence of percolating water and ground-water tensions on the stability is also discussed.

One Paper describes some interesting experimental work carried out in connection with the Fort Peck dam closure-wall in which the shearing-stresses are examined by photo-elastic methods using gelatine models. On the practical side some interesting information is given in a Paper which describes and discusses examples of bank slides on the Whangpoo river in China.

Section H. Bearing-Capacity of Piles.

Papers in this Section are largely mathematical and deal mainly with pile-driving formulas. One Paper, which discusses the recommendations for pile work included in the proposed Boston Building-Code, suggests that a modification of Hiley's formula is probably the most suitable. In the general discussion on this subject two important points were emphasized. One was the importance, in the design of piled foundations, of bearing in mind that, when piles are grouped together, the soil to a considerable depth beneath the piles may be affected, and that a consideration of the geological strata beneath the site is therefore essential. The other concerned the danger which exists in driving piles into certain types of clay. There are certain clays which, though strong in their natural state, become appreciably weaker when their natural structure is destroyed by remoulding, and pile-driving in such clays, causing remoulding, may result in a loss in strength of the clay.

Section I. Pile-Loading Tests.

As may be expected, Papers in this Section consist largely of records of observations taken during the course of construction work involving piles. Records are given of work in China, the Netherlands, Austria and the U.S.A. One Paper deals with an experimental device for estimating the necessary length of bearing-piles and for determining the toe resistance, and its use in connection with foundations of a bridge in the Netherlands is also described.

Section J. Earth-Pressure Against Retaining-Walls, etc.

There are only a few Papers in this Section, and the chief among them are contributions by Professor von Terzaghi. The most important of his Papers is entitled "A Fundamental Fallacy in Earth-Pressure Computations"—a reprint from the Journal of the Boston Society of Civil Engineers.[1] The fallacy he attacks is that the pressure of sand at the back of a lateral support increases like a hydrostatic pressure in direct proportion to the depth. He also discusses the assumptions made in the earth-pressure theories of Rankine and Coulomb in the light of the known relations between stress and strain in cohesionless soils. He concludes that Rankine's assumptions are incompatible with these relations, and recommends that the use of Rankine's theory should be discontinued. In regard to Coulomb's theory, he concludes that this can be used for practical purposes, providing that the lateral yield of the wall exceeds a certain minimum value. This value depends on the height of the fill and

[1] Journal Boston Soc. C.E., vol. xxiii. (1936), p. 71.

also on the nature of the sand, whether it is dense or loose. In another Paper, Professor von Terzaghi discusses the influence of the drainage of retaining-walls on earth-pressure.

Another interesting Paper in this section describes measurements of the distribution of normal pressure on a retaining wall due to a concentrated surface load.

Section K. Ground-Water Movement and Seepage.

These Papers deal mainly with the theory of water movement in soils and its experimental investigation in the laboratory.

Section L. Soil Problems in Highway Engineering.

The importance of a study of the subsoil in connection with road-engineering problems has been recognized in a general way for some time, but it is only during recent years that systematic soil studies have been undertaken. Progress in this direction has been most marked in the U.S.A., but it must be pointed out that much of this work is of a specialized type, for in the U.S.A. soil research for roads is not regarded primarily as an investigation for foundation problems. This is mainly due to the fact that some of their most urgent problems are concerned with earth roads, and the tendency has been to investigate soils from the point of view of their suitability as surfacing-materials in stabilized earth roads. Although, however, a good deal of prominence is given in the U.S.A. to this specialized type of soil research, much work has been carried out in connection with road foundations and road problems of a more general nature. These include such important problems as the design and construction of fills and embankments, the stability of natural slopes and the survey and design of subsoil foundations.

Perhaps the most interesting of the Papers in this Section are those which deal with an aspect of highway engineering, known as the soil survey, which has apparently proved of great practical value in problems relating to both design, construction, and maintenance of roads. So far as the Author is aware, the soil survey has no counterpart in the engineering practice of this country, and it may, therefore, be of some interest to give a brief description of the work it involves and the kind of information it provides.

The soil survey comprises the collection of data on the nature and condition of the foundation soils along a given route. The data are obtained by (1) the examination of the soil profile, and (2) laboratory tests on soil samples. The information thus obtained is mapped and commented upon in a soil-profile report. The soil profile is examined by making borings or test pits at frequent intervals along the route,

and in many cases an area extending 100 feet on each side of the centre line is considered. Borings are taken to a depth which will give information on all the materials which are likely to be affected by construction operations. Records are taken of the depth and extent of the soil strata encountered, and a description is given of the natural soil conditions.

Typical samples of the soil are then examined in the laboratory and, by means of simple empirical tests, are classified into groups according to the methods which have been developed by the Bureau of Public Roads and which have been described in their publications. This information is then incorporated in a soil-profile report, which consists of a map showing the soil strata along the route together with comments on the soil condition and recommendations in connection with various aspects of design and construction. The soil-profile report is submitted to the designing and constructing engineers. In the first place it gives an inventory of the soil resources which is particularly useful in such items as the availability of sands for embankment construction or clay for "shoulder stabilization." It indicates clearly rapid changes in subsoil type which may call for variation in base-course design. It often gives an indication of conditions which suggest costly construction or maintenance, and may facilitate the choice of a more suitable alignment of the road. It also gives a better visualization of cut-and-fill along a proposed route, and enables advantage to be taken of good natural foundation materials. For instance, if the foundation soil is good, heavy cuts with the side slopes moved back may be recommended, or if the soil is of such a nature that excavation is undesirable, heavy cuts are avoided. It may also assist considerably in the location of under-drains, and some idea of the effectiveness of drainage can be estimated. Exceptional soils may be encountered which, if moved during a rainy season, may become unstable, whereas construction with the same material in a dry season would offer no special difficulty. A knowledge of these facts permits work to be performed during the most favourable season.

To summarize these points, it appears that the data derived from soil surveys are considered of important practical value, and their use in America is stated to have developed new trends of practice and opened up new fields of thought in highway engineering. Among these may be mentioned (1) the replacement of a uniform base course with a varied design; (2) correct location of under-drains; (3) the abandonment of the policy of balancing cut-and-fill sections for the more intense utilization of good natural foundation materials; (4) the removal of material likely to cause excessive frost-heave; (5) special methods of dealing with swamp areas.

In connection with this last item, one Paper describes some interesting work in connection with the construction of a fill over a mud flat in California. This soil was of a very compressible type and in previous constructions much difficulty had been experienced owing to the protracted period over which settlements occurred. Tests were therefore carried out to investigate the effect of vertical sand drains in accelerating the consolidation of the subsoil. Laboratory tests indicated that a specimen with vertical sand drains consolidated nearly twenty times as fast as the normal specimen. Practical tests were then carried out on a selected site and eighty-four holes, each of a diameter of 28 inches and spaced from 10 to 12 feet apart, were bored to an average depth of 42 feet and back-filled with clean river sand. These tests showed definitely that the method did accelerate consolidation considerably and that the fill became stable in a period varying from 6 months to 1 year. Experiments are being continued to inquire into the economic aspects of the method.

Another Paper describes work in connection with a similar problem in Holland. In this case a brushwood "mattress" of special construction was employed to carry the road and apparently it functioned satisfactorily. At one of the sessions of the Conference an interesting lecture was given by Dr. Loos of the *Technische Hochschule*, Berlin, on the different methods which have been tried in Germany for the rapid consolidation of fill-material in connection with the construction of motor roads in that country.

In conclusion, the following list of some of the problems which are under investigation in the U.S.A. may give an idea of the extent to which soil research is being applied :—

(1) Study of foundations for bridges and embankments.
(2) Investigation of materials for embankment-construction.
(3) Drainage properties and requirements.
(4) Frost-heaving and its prevention.
(5) Investigation of soils on which concrete pavements have warped.
(6) The proper construction of base courses and subgrade treatment.

One point which may be emphasized is that in all this research the importance of systematic observations in the field is fully recognized, and it is significant that one aspect of the soil organization in U.S.A. is the training of highway engineers in the elements of soil mechanics and soil testing.

Section M. Methods of Improving the Physical Properties of Soils for Engineering Purposes.

The Papers in this Section deal mainly with practical processes employed to improve poor foundation soils. One Paper describes the use of bituminous emulsions in Holland; another outlines cement-injection methods used in France; and another gives an account of a chemical treatment of the soil at a site in Italy. One Paper of theoretical interest describes laboratory tests in which clay is hardened by an electro-chemical process using an aluminium anode.

Section N. Modern Methods of Design and Construction of Foundations.

The Papers in this Section are primarily of practical interest and describe methods adopted to overcome adverse foundation conditions. It is important to note, however, that in most cases the work was accompanied by thorough subsoil exploration and that many of the methods constitute interesting examples of the direct application of knowledge derived from soil tests.

A particularly interesting example is one of the design of a rigid heavy foundation for the Palace of the Soviets in Moscow. A thorough examination of the site was made and consolidation tests were carried out on soil samples. As a result of the knowledge thus gained a design was evolved which saved both time and money and also reduced the hazards involved in installing the footings.

Another interesting example is that of a building in New York where, by using the results of soil tests, a foundation problem was solved in a manner differing radically from usual practice. At this site it was concluded from soil tests that differential settlement could not be avoided, but it was possible to predetermine its magnitude. In the light of this knowledge the basement of the building was constructed as a reinforced-concrete rigid frame and this has functioned satisfactorily.

The practical utility of soil tests is also referred to in some interesting Papers dealing with the construction of bridge foundations in New Orleans and in Denmark. In the New Orleans bridge, settlement calculations based on soil tests were made for all piers. These were checked during construction and, as a result, useful modifications were introduced into the completed bridge design. In the Danish bridge soil tests and observations were found of great assistance during construction.

Section Z. Miscellaneous.

In this Section mention may be made of several Papers dealing with the application of soil mechanics to earth-dam construction.

A great deal of construction work of this kind is being carried out in the U.S.A. at the present time, and it involves considerable expenditure. Among the larger projects may be quoted the Muskingum Valley flood-protection works; Fort Peck dam, described as the largest hydraulic-fill dam in the world; and the Quabbin reservoir for the Boston Metropolitan District Water Supply. In all these projects soil tests are being used in a large measure as a control during construction.

Another interesting example of the importance which is being attached to soil-mechanics research is contained in a Paper which presents a summary of the chapter on foundations which is included in the proposed new Boston Building-Code. It is noteworthy that for the compilation of this chapter the committee, consisting of architects, engineers, builders and others interested, was augmented to include a number of soil experts. The chapter on foundations is different from that of other building-codes since it reflects in some measure the new ideas of soil mechanics. In many cases the clauses represent a compromise between old and new conceptions in the field of foundation engineering, but the chapter represents an attempt put foundation codes on a more scientific basis.

General Review.

In making a general review of the results of the First International Conference on Soil Mechanics and Foundation Engineering, it is possible to draw a number of important conclusions:—

(1) That the importance of soil studies for foundation purposes is recognized in nearly every country in the world, and that in many countries research is being very actively pursued.

(2) That, although there is still considerable leeway to be made up on the fundamental side, the research has progressed sufficiently to allow the standardization of a number of experimental methods.

(3) That one of the most urgent needs at the moment is a more active co-operation between the practising engineer and the soil worker.

(4) That already the application of the knowledge derived from soil research has proved of value in engineering practice, and that in a number of countries it is regarded as essential in dealing with foundation problems.

In conclusion, it seems that the importance of this new branch of science is likely to attain more widespread recognition, and, since its field of application is so extensive, its growth is bound to exert a profound influence on engineering practice of the future.

RESEARCH WORK IN ENGINEERING AT THE CITY AND GUILDS (ENGINEERING) COLLEGE, OCTOBER, 1936.

It is proposed to give a brief description of the research work being carried out in the Departments of Civil, Mechanical, and Electrical Engineering of the City and Guilds (Engineering) College of the Imperial College of Science and Technology, London University.

Department of Civil Engineering.

Research in this department is being carried out on structures, hydraulics, and highways.

On the structural side considerable attention is being given to the mechanical analysis of stress and many questions arising in structural design are being investigated by means of models. A research into the problem of the voussoir arch is in progress at the request of the Building Research Board, and the first part of this work, dealing with the mechanics of the problem, is just approaching completion. The investigation has been made by using an arch-model constructed of steel voussoirs, and interesting evidence on the behaviour of such structures has been obtained. The work is being extended and it is hoped that it will produce results of direct utility in design.

A simple method for determining influence-lines without the use of elaborate apparatus has been developed, and is described in the present number of the Journal.[1] An instrument, known as the "force balance," has been designed for determining the end bending moments on encastré structures. A small model of the structure to be examined is made of duralumin and appropriate loads are applied. The end bending moments are measured and it is found that the results agree very closely with those obtained by calculation. An investigation of spandrel-braced arches is being carried out by model-methods, using Beggs's apparatus in combination with mathematical analysis.

The problem of two-dimensional stress-analysis has been attacked on the experimental side, using a model made of thin sheet india-rubber. The model is kept flat by floating it on mercury, is suitably loaded, and its deformations are measured by a micrometer microscope. From these measurements the stresses can be calculated,

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[1] Prof. A. J. S. Pippard and S. R. Sparkes, "Simple Experimental Solutions of Certain Structural Design Problems." Journal Inst. C.E., vol. 4 (1936–37) p. 79. (November, 1936.)

and the results have been found to agree remarkably well with theoretical prediction. This work has been recently described.[1]

Recently, experiments have been made to determine the stress distribution in the web of a riveted plate-girder with special attention to the effect of variations in spacing of the stiffeners. The results were found to agree well with theoretical investigation except in the immediate neighbourhood of the applied loads. An electric-arc-welded five-panel Vierendeel truss, constructed in the laboratory, has been subjected to an elaborate investigation to determine the stress distribution both in the main members and in the actual joints. The agreement between experimental results and those obtained by various theoretical methods is very good, especially when, in the analysis by deflection methods, allowance is made for the effect of shearing forces. Experiments on welded angle-bar tension members have shown that a welded member should be subject to the same limitations regarding net effective sectional area as a riveted one. A very high stress concentration was disclosed at the roots of the angles. An investigation has recently been carried out on the behaviour of reinforced-concrete beams with a view to determining the true value of the flexural rigidity for use in calculations.

Research in the Hydraulic Laboratory falls naturally into two categories. On the one hand there is long-range research whose aim is to study and develop theoretical methods and experimental technique. On the other hand there are problems of immediate engineering importance to be solved, and for this reliance must be placed upon the outcome of long-range researches by earlier workers.

An investigation of flow past roughened plates has just been completed, the aim being to correlate the frictional effects with those occurring in flow through pipes. The numerical data obtained extend the range of Froude's classical work on towed planks, while the constants have been correlated with those found by Nikuradse for pipes. These results have already been utilized in another research on discharge coefficients of broad-crested weirs, for which frictional effects can now be predicted. In these weir experiments some fifteen different forms of crest were studied.

Experiments with small models of spillway-dams and weirs have thrown light on the causes of scour in the river bed below a dam. Many alternative forms of protective toes have been compared, and the conditions under which each type is most successful have been found. Another investigation deals with silt transport and scour in canals and rivers; this is being attacked by a study of the motion

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[1] L. Chitty and A. J. S. Pippard, "On an Experimental Method for the Solution of Plane Stress Problems" Proc. Roy. Soc. (A), vol. 156 (1936), p. 518.

of silt particles in a turbulent fluid, and these results are being checked against a model river with a bed of loose granular alluvium. Experiments are also in progress on the erosion of a sand-bed by jets of water and of air.

The resistance laws for flow in open channels have been studied by experiments with artificially-roughened beds. A recent research on pipe friction has shown why practical pipe-lines differ from those of the laboratory. In these experiments different types of roughness were simulated by various arrangements of sand grains fixed to the inner wall of a pipe. Loss of capacity with age is being studied by an analysis of existing pipe-line data, from which it appears that the annual deterioration due to incrustation can be predicted by a simple, though rational, method.

Flow through nests of tubes is being tackled directly by measurements of friction and pressure distributions, and indirectly by sectioned models in which the motion is made visible by aluminium powder sprinkled on the water surface, the ultimate aim being to find the optimum spacing and arrangement of the tubes for low friction and high heat-transfer. Another method of flow-analysis, in which an improved technique is being developed, makes use of the analogy between electrical and hydrodynamic flow. Fluid-friction effects have been included in the electrical model, and practical streamline and pressure distributions can now be very rapidly obtained. An open-jet type water-tunnel is being used to study cavitation caused by bodies of various shapes and materials. A research with small models is being carried out for The Institution to determine the most suitable form of fish-pass. In a research on transportation of sand by wind, studies are being made of the formation of sand dunes, sand storms and other desert phenomena.

In the Highway Laboratory research is being made on soil physics, and methods of sampling and analysis of soils have been evolved whereby the suitability of the sub-soil as a road foundation may be assessed. In view of the increasing importance of the subject in colonial development, the stabilization of earth roads by oils and bituminous emulsions is being investigated. Tar and bituminous surfacings usually fail by the formation of corrugations and a research has been made into the best methods of testing a bituminous mixture in order to give a measure of its resistance to corrugation. In addition to research on bituminous carpets, the use of rubber in various forms and mixtures has been investigated. The adhesion between filler and matrix is found to be of great importance in the production of a stable carpet. Another research is being made into the conditions affecting the slipperiness of road surfaces. A pneumatic-tired wheel is mounted so as to skid on a rotating glass

surface, and the effect of water, oil, and various mixtures is being studied, while later it is hoped to extend the research to rough surfaces.

Department of Mechanical Engineering.

A large amount of research is being conducted on heat-transmission. In a research on the radiation from non-luminous gases where the heat-transmission from the products of combustion of ordinary town's gas with temperatures between 1,000° and 2,000° F. is measured, many interesting results have been obtained. It has been found that the carbon dioxide and the water-vapour constituents alone radiate heat, and the experiments have shown the predominating importance of this radiation over convection in the dissipation of heat at such temperatures.

A general study is being made of convection in gases. This is of great importance in connection with the thermal properties of buildings, furnaces, and the like. The conditions obtaining with large heated surfaces can be faithfully reproduced in a small-scale model by the use of a high air-pressure. In this way a study on the flow of air past a heated vertical plate has been made. Such a flow is laminar up to a certain point beyond which it becomes turbulent. In addition to plane surfaces, cylinders and parallel plates have been investigated. In an allied research the loss of heat by convection from a heated wire is studied in various media and at various pressures. Pressures up to 1,000 atmospheres have been employed. In this way it has been possible to study the thermal properties of gases at such high pressures, and information has been obtained on the relations between thermal conductivity, viscosity and pressure.

An optical method of studying heat-transmission problems has been developed and has proved very useful. At a sufficiently small distance from a heated surface convective circulation is very small and a fairly steady temperature-gradient exists normal to the surface. This produces a density-gradient in the fluid, and in consequence a beam of light grazing the surface is bent outwards by an amount which gives a measure of the temperature-gradient and thus of the transfer of heat. The method has been applied to a heated horizontal flat plate in air. On the underside a condition of stable equilibrium obtains, but on the upper surface instability occurs at a certain calculable temperature-gradient and turbulent motion ensues. The effect of the size of plate has also been studied. The method is also applicable to water. Under an increasing temperature-gradient a layer of water between two horizontal flat plates is observed to pass successively through three different states, laminar, cellular, and turbulent motion respectively.

When a fluid is at a different temperature from that of a surface, the rate of transfer of heat with forced convection is found to depend upon the frictional resistance or roughness of the surface. Surfaces with exaggerated roughness of various forms, grooves, ribs and projections, and with air-currents inclined at various angles to their direction have been studied. With asymmetrical grooves the direction of flow was found to be of importance. This has been illustrated by the flow of water past a large-scale model of the irregularities, aluminium powder being sprinkled thereon to show the motion.

Among other researches may be mentioned an investigation into the mechanical properties of frozen mercury at various low temperatures. The effect of rigid joints on the stress in a model airship-frame is being investigated experimentally and the results checked against theory. It is found that while the direct stresses are little different from those obtained in the corresponding pin-jointed structure, comparatively large stresses due to bending are present at the joints.

Department of Electrical Engineering.

Much of the electrical research concerns high-frequency measurements. With the increasing importance of television it is necessary to evolve suitable measuring instruments. A screened hot-wire ammeter has been constructed in which the hot wire is mounted co-axially in a screening tube. With this simple geometrical arrangement it is possible to calibrate the hot wire by direct current and to apply a calculable correction from the known geometrical properties of the screening tube and the capacity of the terminals. Some experimental work has also been carried out on a high-frequency Wheatstone bridge. The properties of the high-frequency thermionic peak voltmeter are being investigated both theoretically and experimentally. This has been done by comparison with the voltage induced in a hot wire, which can be calculated from its extension, and it is found that the accuracy of the peak voltmeter is impaired both by resonance and by the time of flight of electrons in the valve. A research has just been completed on the response curves of electric filter circuits. The relation between energy-output and frequency is recorded in graphical form by a cathode-ray oscillograph. Another research which has recently been completed is an investigation of the power factor at high frequencies of insulating materials. The results obtained were in close agreement with an independent investigation on the same lines which was carried out at the National Physical Laboratory. Further developments of the methods of high-frequency measurements now depend upon the making of

oscillators capable of dealing with a considerable current, and this subject is now being studied.

Among researches which have been conducted into problems concerning electrical machines may be mentioned a study of the increase of contact resistance in the collection of current from slip-rings at high speed of rotation and the effect thereon of pressure of contact and smoothness of surface.

An acoustic research has been instituted and noise-proof rooms have been prepared in which the transmission of noise may be studied and measured. Electrical means for the control of reverberation are being evolved.

A research on high-voltage surges, in which, by means of a klydonograph, a study is made of spark discharge, has produced some as yet unsolved problems; these include the determination of the relative importance of the maximum value and the duration of the surge in the production of the klydonograph figure, a problem involving the measurement of the time of breakdown of a spark-gap.

The foregoing researches are being carried out under the direction of Professor C. H. Lander, C.B.E., D.Sc., Professor of Mechanical Engineering and Dean of the College, Professor A. J. S. Pippard, M.B.E., D.Sc., Professor of Civil Engineering, Professor C. L. Fortescue, O.B.E., M.A., Professor of Electrical Engineering, and Professor R. G. H. Clements, M.C., Professor of Highway Engineering.

REPORT OF THE FOREST PRODUCTS RESEARCH BOARD FOR THE YEAR 1935.

The Report of the Forest Products Research Board of the Department of Scientific and Industrial Research, 1935, covers a wide field of research and records many interesting results. A study has been made of the relation between the structure and the technical properties and mechanical strength of wood, and it is found, for example, that in the case of ash the butt logs yield the toughest wood, but the upper logs are strongest in compression parallel to the grain. With timber there is, in general, a large variation between experimental results on apparently similar specimens and wide use is therefore made of statistical methods. An accurate measurement of cell-space ratio is now possible by photo-electric means. An optical method has been evolved whereby the smoothness or finish of planed wood is measured by the degree of scattering of a parallel beam of light. A study of the moisture-content of wood has shown that part of the water is held by molecular absorption and part by capillary action. The relation between moisture-content and the

surrounding hygrometric conditions, including a general study of seasoning processes, indicates that after drying the capacity for holding moisture is impaired. The equilibrium moisture-contents of timbers stored under various conditions are given. It is of particular interest to note that while the moisture-content of timber for a building may be as much as 20 per cent. when built in, it reaches an equilibrium moisture-content of about 12 per cent. Shrinkage, wood-bending and warping, tests of timbers of structural sizes and the effect of size, resistance to abrasion and tests for durability are discussed. Home-grown timbers have received particular attention, and the application of wood-preservatives and a general study of rot and wood-borers has been made.

NOTES ON RESEARCH PUBLICATIONS.

Measurement and Measuring and Recording Instruments.

The engineering applications of the Poisson probability summation and the treatment of sampling and related problems thereby are dealt with in *Jun. Inst. Engrs. J.*, **4**, *479*. A method of determining the percentage of moisture in soil samples without drying is described in *Am. Soc. Test. Mat. Bull. 1936* (*81*) *10*. An adaptation of the cathrode-ray oscillograph for the simultaneous recording of two phenomena is described in *Elek. und Masch.*, **59**, *409*. A method for determining the velocity of sound in solid rods by measuring the wave-length and frequency of vibrations is explained in *Rev. Scient. Instruments*, **7**, *252*.

Engineering Materials: Properties and Testing.

Timber.

Notes on the shrinkage of wood are contained in *J. and Proc. Roy. Soc. N. S. Wales*, **69**, *174*. The properties of the Malayan timbers Seraya, Meranti and Lauan, are described in *Research Record No. 12*, issued by the Forest Products Research Laboratory of the Department of Scientific and Industrial Research.

Cement and Concrete.

The distribution of compounds in portland cement is discussed in *U.S. Nat. Bur. Stand. J. Research*, **17**, *261*. A study by X-ray analysis of the crystalline forms of cements during thermal treatment is described in *La Chimica e l'Industria*, **18**, *277*. Some comparative tests on the effect of low temperatures upon cements, particularly

supersulphated metallurgical cements, are discussed in *XV^me Congrès de Chimie Industrielle* (*Brussels 1935*), *Compt. Rend.* **1,** *364.* An investigation on the use of high-early-strength cements in concrete products is described in *Am. Concr. Inst. J.*, **7,** *673.* In *XV^me Congrès de Chimie Industrielle* (*Brussels 1935*), *Compt. Rend.*, **1,** *441*, the *p*H-value of the mixing water is shown to have an effect on the setting time of various hydraulic cements: the more alkaline the water the shorter is the time of setting. The hardening of concrete and the variation of its strength with absolute volume of aggregate and with water/cement ratio is dealt with in *Bull. Tech. de la Suisse romande*, **62,** *53 and 61.* The optimum concrete mixes for dams and other mass concrete work are discussed in *Beton und Eis.*, **35,** *18.* The technique employed and the results obtained in testing the permeability of concrete slabs are described in *Zement*, **25,** *392.* The composition and results of recent tests on fat lime mortar and concrete used in the construction of a dam in 1865 are given in *Ann. Inst. Tech. Bâtiment*, **1,** *9.* An article on magnesium oxysulphate cement is contained in *XV^me Congrès de Chimie Industrielle* (*Brussels 1935*), *Compt. Rend.*, **1,** *485.*

Metals.

The apparent impact bending strength of a specimen is stated to be in excess of the true impact strength owing to the kinetic energy imparted to the broken fragments, and the extent of correction necessary is given in *Deutsche keramische Gesell. Berichten*, **17,** *281.* The date and duration of expansion of metals under static tension is dealt with in *K. W. Inst. Eisenforsch.*, **18,** *51*, and in the same journal *p. 65*, is an article on stress conditions during forging. The strength and elastic properties of cast iron are discussed in *Iowa Eng. Expt. Stn. Bull.*, *No. 127*, its modulus of elasticity in *Rev. Métall.*, **33,** *498*, and the influence of surface condition upon its physical properties in *Giesserei*, **23,** *466.* Work at the National Physical Laboratory is described in the following papers in the *Journal of the Iron and Steel Institute*, **133**: *p. 303P*, The behaviour of five cast irons in relation to creep and growth at elevated temperatures; *p. 399P*, Further experiments on the effect of surface conditions on the fatigue resistance of steels; *p. 427P*, Internal stresses and their effect on the fatigue resistance of spring steels. The applications of low-alloy steels are discussed in *Welding J.* (*New York*) **15,** *2.*

The Fourth Report of the Corrosion Committee of the Iron and Steel Institute for the year 1936, giving the present position of this research, has been published. An article on the resistance of steel-copper alloys to corrosion appears in *Rev. Univ. Min. et Mét.*, **12,** *365.*

The effect of repeated impact on light metals is discussed in *Zeit.*

Metall., **28,** *233.* A method of analysing creep data is given in *J. Applied Mechanics*, **3,** *A62*, and the creep of tin and tin alloys, describing work carried out at Birmingham University, in *Inst. Metals J.*, **59,** *183.*

Other Materials.

Methods adopted in studying the physical properties of coal tars are reviewed in *Indust. and Eng. Chem.*, **28,** *721.*

ENGINEERING MATERIALS: PRODUCTION, MANUFACTURE, AND PRESERVATION.

Brick and Concrete.

Research on the use and grading of stone sand for concrete and for asphalt is described in *Bull. No. 10* of the National Crushed Stone Association, Inc. (Washington). A German standard specification for sand-lime bricks has been published by the *Deutscher Normen-Ausschuss, Berlin, 1936.* A draft specification for aggregates and gauging water for use in the preparation of concrete is given in *Ann. Inst. Tech. Bâtiment*, **1,** *67.* A new type of asbestos-cement product involving the employment of siliceous or pozzuolanic admixtures is described in *Kolloid Zeit.*, **75,** *354.* The resistance of concrete masonry walls, with various surface treatments, to the penetration of rain is dealt with in *Am. Concr. Inst. J.*, **7,** *485.*

Metals.

The dephosphorization of steel by means of alkaline slag in the coreless induction furnace is discussed in *Stahl und Eis.*, **56,** *1179.*

STRUCTURES.

Mass Structures.

The following tentative test methods on soil mechanics are given in *Am. Soc. Test. Mat. Tentative Standards 1935*: *889*, Method of mechanical analysis of soils; *901*, Method of test for liquid limit of soils; *905*, Method of test for plastic limit and plasticity index of soils; *908*, Tentative method of test for centrifuge moisture equivalent of soils; *911*, Tentative method of test for field moisture equivalent of soils; *913*, Tentative method of test for shrinkage factor of soils. A method for determining the amount of entrapped air in capillary soils is given in *Eng. News-Record*, **117,** *186*, and on *p. 194*, a method of testing soils for foundations by the application of a horizontal test load to the walls of a trial pit by means of a jack is described.

Framed Structures.

The theory of elastic arches is given in *Ann. Lav. Pubblici*, **74**, *503*. A method for the analysis of the stability of rectangular plates elastically supported at the edges, in which use is made of standard curves, is given in *J. Applied Mechanics*, **3**, *A47*, and on *A55* is an article on impact on beams. A method of analysis of continuous frames by balancing angle changes at the joints is described in *Am. Soc. Civ. Eng. Proc.*, **62**, *995*. On the subject of earthquakes: a paper on the design of structures to resist earthquake forces is given in *Commonwealth Eng.*, **23**, *363* ; a book on earthquakes and design of structures, by J. Bakshi (*Cuttack, 1935*), draws conclusions from the Indian earthquake of January, 1934 ; a survey of type failures of buildings resulting from earthquakes at Palo Alto, California, is given in *Building Standards Monthly*, **5**, *4 and 14* ; the energy-dissipation in seismic vibrations of actual buildings of unlike structures is discussed in *Bull. Earthquake Research Inst., Tokyo*, **14**, *119*, and on *p. 134* the energy dissipation in seismic vibrations of a six-storeyed structure is discussed. The effect of length and percentage of reinforcement on the initial stress in reinforced concrete due to shrinkage is dealt with in *Beton und Eis.*, **35**, *14*, and on *p. 29* the old and new modular ratios used in the design of reinforced concrete beams are discussed. The Indian Railway Standard code of practice for reinforced-concrete construction has been published by the Government of India (Railway Board).

Constructional Operations and Methods.

An electrical treatment of concrete during deposition to prevent freezing is described in *Gén. Civ.*, **109**, *184*, and on *p. 263* of the same journal a method of testing the soundness of concrete piles during driving is given.

Transformation, Transmission and Distribution of Energy.

Research on gas-purification by the process employing iron oxide is described in *Gas- und Wasserfach*, **79**, *705*. Research on internal-combustion engines includes : an analysis of the actual performance of petrol engines carried out at the National Peiyang Eng. College, *Engineering Research Inst. Memoirs No. 2* ; the use of coal-tar oils in high-speed diesel-engines, *Glückauf*, **72**, *697* ; and a research on flame-movement and pressure development in an engine cylinder, described in *Nat. Adv. C. Aer. Report 556*. The following electrical researches have been noted : the electrical stability of condensers,

Inst. Elec. Eng. J., **79,** *297* ; recent investigations on electric arcs : the mercury-arc generator, *Hochfrequenz.*, **48,** *22* ; investigations on impulse stresses in transformers, *Archiv Elek.*, **30,** *368* ; electrical conductivity of cast iron, *Zeit. Metall.*, **28,** *224*; and the effect of earth resistance of transmission-line poles upon lightning discharges, *Elek. Zeit.*, **57,** *1079.* The dynamic balancing of ship propellers, in which the practical aspect is dealt with, is given in *Am. Soc. Naval Eng. J.*, **48,** *327*, and on *p. 361*, noise and vibration in naval vessels are discussed.

Mechanical Processes, Appliances and Apparatus.

Research on the metallurgy of " pure " iron welds carried out at Lehigh University is described in *Am. Welding Soc. J.*, **15,** *Research Supplement, p. 5*; and in the same supplement, *p. 13*, the effect of weld penetration on stresses in fillet-weld joints, as ascertained from models by photo-elastic methods, is discussed. The present position of research on surface films and lubrication is described in a paper in *Roy. Aero. Soc. J.*, **40,** *754.*

Specialized Engineering Practice.

Transport.

The incorporation of resinous products in hydrocarbon binders is shown to improve the adhesion where the aggregate is of a calcareous nature, in *Rev. Gén. des Routes*, **11,** *327.* The effect of various finishes to asphalt roads is described in *Mitteilungen der Forschungsgesellschaft für das Strassenwesen, Berlin 1936, (8), 65.*

Researches dealing with railways include a method of testing the air resistance of locomotives, *Rev. Gén. Chemins de Fer*, **55**-*ii*, *35* ; Trials to determine the effect on tractive effort of gauge widening on curves, *Quarterly Technical Bull. Indian Ry. Board* **4** (*42*) *1* : and the Second Progress Report of the joint investigation of fissures in railroad rails, *Univ. Illinois, Eng. Expt. Stn. Bull.* **34,** *No. 19.*

An experimental study of the scour of a sandy river-bed by clear and by muddy water is described in *U.S. Nat. Bur. Stand. J. Research*, **17,** *193.* The results of model tests on ship forms of varying fulness are given in *Schiffbau*, **37,** *285.*

Air transport is dealt with in *Roy. Aero. Soc. J.*, **40,** *563*, where an account is given of recent developments in the theory of the boundary layer, and in the same volume, *p. 769*, are given formulas and methods of calculation of the strength of plate and shell structures in aeroplane construction. Full-scale tests of landing flaps on a Percival " Gull "

aeroplane are described in *Aer. Research Committee Reports and Memoranda, No. 1697*, and in No. *1689* is discussed the contribution of the body and tail of an aeroplane to the yawing moment in a spin. The influence of tip shape on the marginal phenomena in aeroplane wings is dealt with in *Compt. Rend.*, **203,** *489*. Researches on the helicoplane are described in *Aéronautique* (*Aerotech*), **18,** *88*; skimming surface tests at high Froude numbers and the aerofoil comparison in *Luftfahrt.*, **13,** *269*; wind-tunnel research on the air resistance of air-cooled radial aircraft motors, *Luftfahrt.*, **13,** *239*; air-flow around finned cylinders, *Nat. Adv. C. Aer. Report No. 555*; and a contribution to the design of axial-flow propeller-type machines with their housing, *Kyushu Imperial Univ., Japan, Memoirs Fac. Eng.*, **8,** *91*.

Water-Supply and Irrigation.

The flow of water is dealt with in the following: Uniform flow in weldless steel pipes, *Ann. Lav. Pubblici*, **74,** *493*; Loss of head in tunnels and pipe-lines, *Elec. World*, **106,** *2201*; Analysis of flow in networks of conduits or conductors, explaining a rapid method for approximate calculation of the flow in pipe networks, *Univ. Illinois Eng. Expt. Stn., Bull. No. 286*. Experiments on the hydraulics of irrigation siphons placed transversely over dykes are described in *Magistrato alle Acque, Pubblicazione No. 138*.

Mining.

An article on the composition and fineness of powdered materials used for stone-dusting in coal-mines, based on work carried out at the Mining Research Laboratory of the University of Birmingham, is given in *Inst. Mining Eng. Trans.*, **91,** *391*.

Lighting, Heating, and Acoustics.

The development of the high-pressure mercury-vapour lamp in public lighting is traced in *Inst. Elec. Eng. J.*, **79,** *241*. The results of research on the heat-losses through tiled roofs, carried out at the Building Research Station, are given in *Inst. Heating and Ventilating Eng. J.*, **4,** *313*. An article on the heat insulation of organic building materials under working conditions, describing experiments on the variation of heat conductivity with moisture-content with various materials, is given in *Gesundheits Ing.*, **59,** *261*. The sound-absorbing value of Portland cement concrete is dealt with in *Am. Concr. Inst. J.*, **7,** *659*, and the effect of an acoustically absorbent lining upon the sound-insulating value of a double partition is described in *Proc. Phys. Soc.*, **48,** *690*.

Miscellaneous.

A note on fracture, including a critical analysis of present theories, is given in *Proc. Roy. Soc. Edinburgh*, **56**, *158*. A solution of the three-dimensional elasticity equations for the case of a force at a point in the interior of a semi-infinite solid is given in *Physics*, **7**, *195*. In *Phil. Trans. Roy. Soc., Series A*, **235**, *415*, is a paper on time lag in a control system.

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